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REPORT

ON

NEW YORK'S WATER SUPPLY

1900

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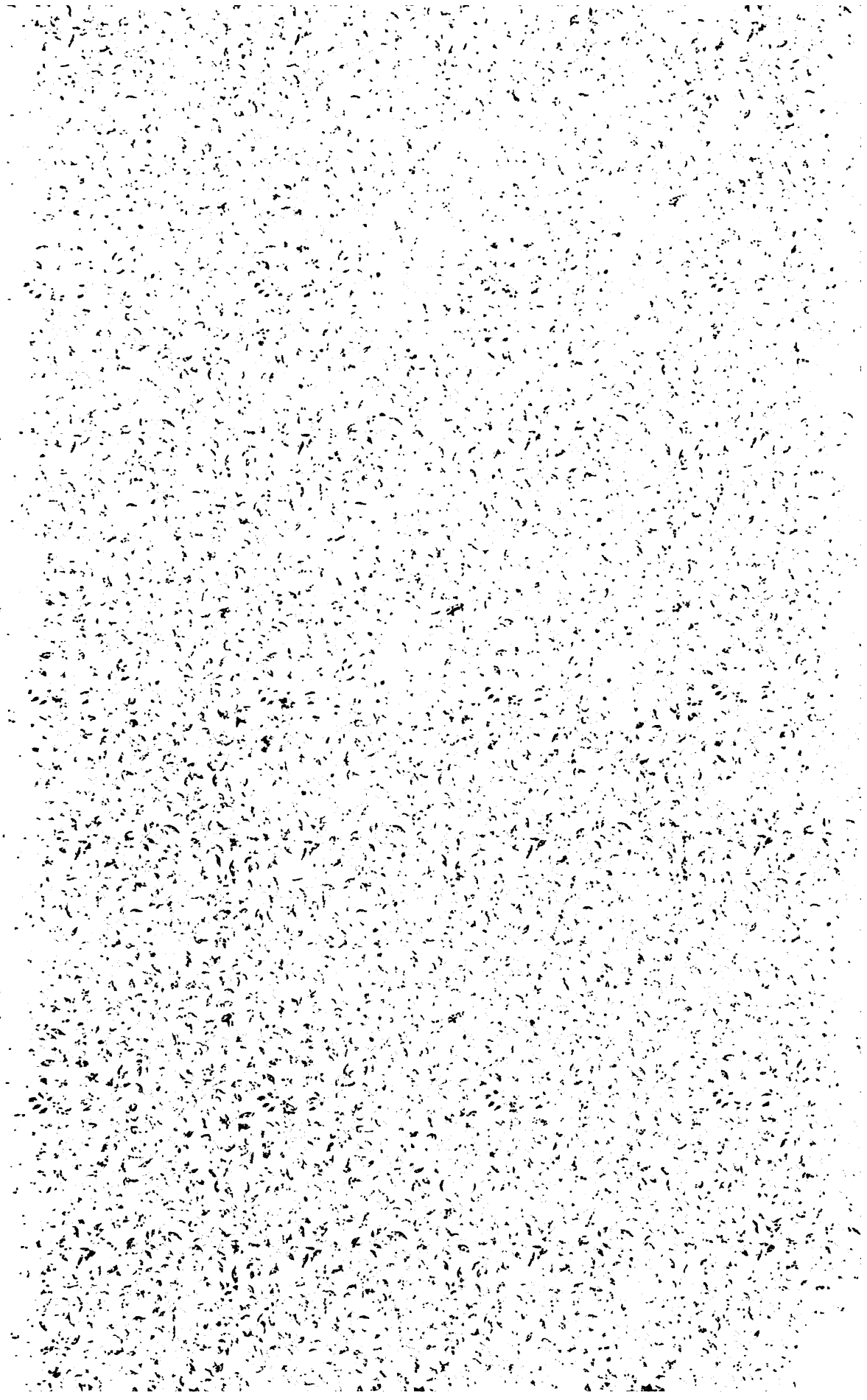
THE GIFT OF

GEORGE GRAFTON WILSON

Professor of International Law



University Club
Providence

Sat 6-15 Pm

My dear Wilson

We have special treat tonight - Prof Sedgwick has brought Prof Underwood the great mosquito hunter!, along with 50 ± of his best lantern slides Prof S. was one of the leaders in the Boston School improvement agitation and this has drawn Edwards to the dinner against a counter attraction and I hope you may be equally susceptible to attraction

Can come
a place at table is saved for you

J R Freeman

REPORT

UPON

NEW YORK'S WATER SUPPLY

WITH PARTICULAR REFERENCE TO THE

NEED OF PROCURING ADDITIONAL SOURCES AND THEIR PROBABLE COST

WITH WORKS CONSTRUCTED UNDER MUNICIPAL OWNERSHIP

MADE TO BIRD S. COLER, COMPTROLLER

BY JOHN R. FREEMAN, CIVIL ENGINEER

MARCH 28, 1900

(With notes added while revising proof-sheets)

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PROVIDENCE, March 23, 1900.*

BIRD S. COLER, *Comptroller*,
Department of Finance,
New York City:

SIR—I have the honor to make the following report in reply to your oral request of August 24, 1899, that I investigate the present condition of the water supply of New York City with particular reference to the following points:

First—Capacity and adequacy of the present sources of supply compared with the consumption at present and in the near future.

Second—Amount of waste and extent to which it is practicable to prevent this waste, and thus keep the draft for a long time to come within the capacity of the sources now owned by the City.

Third—Sufficiency of pressure and volume of present water supply for fire protection.

Fourth—Best means of adding to present water resources when occasion requires, with a view even toward the Adirondacks or the Great Lakes for the very distant future.

Fifth—Advantages and disadvantages of the additional source of water supply from Esopus Creek, proposed by the Ramapo Company.

It is but just to yourself, in view of recent events, for me to record your earnest expression to me that you were not asking for evidence to be collected with a view to sustaining your personal views, but that what you wanted was a fair-minded determination of the truth by an engineer familiar with water supply and removed from local prejudices—"to get together as much reliable information on New York's need for more water as is possible in the time available, and present it without reserve."

*With notes added later while correcting proof sheets.

In fairness to myself it must be explained to those not familiar with investigations for water supply, that the time and means at my disposal have not been sufficient for an exhaustive examination of the possible new sources of supply, nor for attaining that degree of precision in the estimates that I would have preferred, and the time has also proved insufficient for a complete study of the Brooklyn and Staten Island water supplies or the manifest defects in the pipe systems; yet, I trust that a positive answer has been found for your fundamental questions and that the evidence collected is sufficient to be clear and convincing upon the following points: In the appendices to the report I have stated data and arguments with more than ordinary fullness in order that the margin of uncertainty may everywhere be clearly seen and that the figures and conclusions may be easily verified or criticized by other investigators in this field.

Conclusions—Stated Briefly.

First—Although the consumption will very soon be uncomfortably close to the safe limit, with prudent management and with moderate and practicable restriction of waste, *the Croton Watershed, supplemented as now by the Bronx and Byram watersheds, can furnish a safe and ample supply for the boroughs of Manhattan and The Bronx for five years to come; thus giving a short but adequate time in which to build works for an additional supply.*

Second—If the public is ready to consent to the complete introduction of domestic meters and if the administration will earnestly and skilfully undertake the saving of water-waste in schools and public buildings and will earnestly and skilfully make such investigation of the locality of the present pipe leakage, as is practicable by closing existing gates and measuring the night flow in various sections by simple means without expensive new appliances; and will then diligently seek and mend the worst of these leaks, at a very moderate total expense, then the safe limit of the present supplies could be extended to 10 years hence, with present rate of growth.

One year is a *very* brief time for the necessary surveys—and five years an almost impossibly short time for building proper aqueducts and dams.

Third—The preliminary steps should immediately be taken toward the acquisition of an additional supply on a comprehensive scale, capable of supplying 150 million gallons at the start, and ultimately yielding at least 500 million gallons per 24 hours (preferably 750), and surveys, borings and river gaugings, with analysis of water samples, by 20 or more field parties, over nearly all the grounds covered in the reconnaissance described in the following appendices, should be begun during the present summer so that con-

struction may be begun, if possible, early in the spring of 1901 or not later than 1902.

While there is ample proof that more than half of the water now delivered by the New York and Brooklyn aqueducts is wasted, the total per capita consumption of water, including waste, in New York and Brooklyn, is to-day smaller than in Boston, Philadelphia, Washington, Buffalo, Albany, Pittsburgh, Chicago, Cleveland, Detroit, Denver, and certain other large American cities, and it appears plain from reasons given later, *that the hope of restricting waste does not justify delay in making sure of an adequate additional supply.*

Fourth—The work of the Water Department in the location and prevention of waste should be reorganized and carried on more skilfully and earnestly than heretofore. The city divided into districts, the statistics of average consumption and night consumption in bulk and per capita obtained and classified. The metered use better classified and the whole statistics published annually in such form that comparisons of economy can be easily and intelligently made.

Fifth—If, from legislative or other causes, there has to be delay in securing a moderate additional supply, then a thorough system of waste-prevention, beginning properly with a meter on every domestic service, *must, of necessity, be begun at once or danger of a water famine invited.*

Sixth—For the proper and reasonable protection of the entire city against fire and for the greater convenience and economy of the householders I would earnestly recommend that all new work in supply and distribution of water tend toward providing a pressure or head of 300 feet above sea level at the city limits (near Yonkers) for any new supply from the north, and (ultimately) a pressure of 100 pounds to the square inch above street level at the City Hall, and that requirements for plumbing and all pipe laying be carried forward with a view to the gradual increase of pressure toward this limit reaching it, say, 15 or 20 years hence. Recent experience in Syracuse proves that such a change of pressure is entirely possible without great disturbance or inconvenience.

Seventh—I find the price proposed by the Ramapo Company exorbitant and that this source, while good in some ways, is less desirable, of smaller capacity and more expensive than other sources that can probably be obtained.

Stating some of the conclusions with a little more detail, we find:

Present Sources, Yield, Use and Waste.

(*A*) The safe yield of all sources at present controlled by the city and available for the supply of the boroughs of Manhattan and The Bronx, consisting of the Croton, Bronx and Byram watersheds, with all present reservoirs and including the Croton and Jerome Park Reservoirs now under construction, cannot be prudently relied upon in years of extremely low rainfall to furnish an average supply of more than 290 million gallons per 24 hours.

(*B*) The average daily consumption of water by the boroughs of Manhattan and The Bronx during the past year (1899) averaged 246 million gallons per 24 hours, and for four years past, has steadily increased at the rate of 15 million gallons per day from one year to the next. At the same rate of increase *the full safe permanent capacity of the above-named sources, with their new reservoirs completed, will be reached in three years, while with extreme care and a moderate amount of waste-restriction they can prudently be made to serve for five years.*

(*C*) The present waste of water amounts to surely one-half, and probably two-thirds of the entire amount of water delivered to the Borough of Manhattan. A large part of this waste is underground, at unknown localities along about 850 miles of pipe and on 120,000 service pipes. Many of the large leaks could doubtless be found and cured by a year or two of systematic work, but the many thousands of small leaks would require many years for definite location and cure, and a small part of this underground leakage is absolutely incurable.

(*D*) With all their waste, New York and Brooklyn now consume less water per inhabitant per day than the other large American cities.

(*E*) Only in proportion to the application of water meters to substantially all service pipes, is it reasonable to expect to obtain any large permanent reduction of the present waste due to poor plumbing and carelessness; and without metering all taps it is impossible to discover the location of the greater portion of the leakage from street mains and defective and abandoned service pipes.

(*F*) Public opinion, as we find it to-day in New York and all the other largest American cities, probably will not sustain the metering of all domestic supplies.

(*G*) It endangers the public welfare to be too hopeful about restricting waste or to expect New York to lead all other large American cities in a great restriction of water-waste, because with these hopes unfulfilled and reservoirs emptied the disaster would then be beyond remedy.

Taking public sentiment and the practical management of water distribution as we find it to-day in New York and in nearly all the other large American cities, and considering that the present supply, with all its waste, costs for all domestic purposes only one dollar per inhabitant per year, and that the utmost additional supply desired can probably be had for 70 cents per present inhabitant per year, the hope of restricting waste does not justify any delay in procuring a large additional supply.

(*H*) The construction of the Cross River Reservoir, in the Croton Watershed now urged by the Department of Water Supply is utterly inadequate for the required increase and more storage in the Croton Watershed is not advisable at present, because proportionately much more water for the money can be obtained as a part of a broader, more comprehensive project. Legislature authority ought to be secured and surveys ought to be begun at once looking toward beginning the work of construction in the spring of 1901 for an additional supply of water for all the Boroughs of Greater New York. The first installation should be at least 150 million gallons daily. The studies for this additional supply should be on a broad, comprehensive plan, seeking sources capable of being ultimately developed to yield 500 million, or preferably 750 million gallons per day, in addition to all that the municipality now owns.

Available Additional Sources.

(*I*) The cheapest and most available source, by far, if the Connecticut Legislature will co-operate is permitting it to be obtained, is the Ten Mile river combined with the Upper Housatonic, which, with no expense for pumping and with comparatively short aqueducts, will deliver 750 million gallons (and possibly 800 million gallons) per day by gravity flow, in the very driest years, at the city limits, from a large new distributing reservoir, at a hydraulic grade of 300 feet above sea level, at a cost, which, if reckoned on the whole quantity, will probably not exceed $10 per million gallons, even if land and water damages are extravangantly high.

This whole 800 million gallons per day of Housatonic water can be delivered in New York for about one-half the total annual expenditure that the 200 million gallons per day from the Esopus, and its connecting water-

sheds would have cost on the terms tendered by the Ramapo Company and with greater certainty of delivery in years of low rainfall, and much greater safety of structures.

(*J*) Various other good water sources can be secured if the offer of a generous purchase price fails to obtain from the citizens of Connecticut rights to divert the Housatonic and Ten Mile rivers.

(*K*) Esopus Creek, the principal source offered by the Ramapo Company for a supply of 200 million gallons per day, if provided with the reservoirs indicated by the location plans filed by the Ramapo Company in the County Clerk's office, and of the sizes and elevations estimated in the report to you by George S. Rice, C. E., August 29, 1899, could alone probably supply in years of extreme low rainfall not more than 100 million gallons per day without reinforcement from other rivers or streams.

From a personal examination of the Esopus dam sites, I find the most of them appear so very unfavorable for high dams and large reservoirs that I consider 100 million gallons per day the very largest safe yield that we are justified in reckoning on as available from the Esopus prior to thorough exploration by deep borings at these dam sites and complete surveys of capacity.

I find every dam site on the Esopus Creek, excepting that for the Olive Reservoir, to be apparently of a very doubtful character in depth of foundation for high dams and in tightness of wings or earth abutments, requiring utmost precaution and very careful, expensive construction, to not endanger the safety of the whole valley below.

By building the dams higher than proposed by the Ramapo Company, if deep borings show this to be safe, by flowing out the thriving village of Shokan and by flowing out the present railroad location, the Esopus might, with great disturbance of population and the flooding of a large proportion of all the good land in the valley, be given storage sufficient at most for a safe yield of 150 million gallons per day, or could, by long and expensive tunnels from the head waters of Esopus to the head of Schoharie Creek, including Batavia Kill, be made to furnish somewhere between 200 million gallons and 250 million gallons per day from these three streams combided.*

(*L*) The cost of a daily supply of 200 million gallons of water from Esopus Creek and its connecting watersheds suggested by the Ramapo Company, if developed by the City under ordinary methods, for all interest, depreciation, care and maintenance, and for a sinking fund which in forty years at 3 per cent. interest would pay off all bonds required for entire cost of construction, turning the whole over free and clear to the City in excellent repair, would be probably less than $30 per million gallons, which is less than one-half the price stated in the proposed Ramapo contract, under which the City would possess neither the works nor a contract for their water at the end of the forty years.

* While Esopus water is nine times out of ten found clear and sparkling, it is very turbid in the spring freshets and after rains from the wash of red clay which is very slow in subsidence. The stream now being rapid quickly runs clear.

Omitting the sinking fund provisions and thus performing precisely the service proposed by the Ramapo Company with works built by the City itself, it would probably cost the City only about $20 per million gallons for this Esopus and adjacent water for which $70 per million was to be charged.

Water supply developed on the large scale, with present low rates of interest, from any favorable source is wonderfully cheap.

(*M*) While the Legislature of the State of New York cannot give to your City the right to divert the water of the Upper Housatonic, and perhaps not the full rights to divert the Ten Mile river, the ease with which similar questions have been settled in the case of the diversion of a portion of waters of the Nashua river from New Hampshire for an addition to the supply of Boston, and the promptness with which rights were secured and settlement made for the diversion of a portion of the waters of the Blackstone river from many Rhode Island mill sites, for the public water supply of Worcester, Mass., gives confidence that these rights can be secured on the basis of ordinary business procedure, by paying generously, and, on the whole, come out nearly as well as in seizure under eminent domain. It is also believed that constitutional authority can be found for this, and that in a neighborly spirit the State of Connecticut will grant any power needed for New York to complete and perfect its title without hindrance from any few adverse interests, so soon as full and generous payments for all injuries to the property or prospects of its own citizens are made secure. Possibly, also, terms could be arranged, fair and advantageous to both States, by which a transfer of the State boundary line could be made, such that the proposed Housatonic Reservoir would lie within New York, much as Rhode Island and Massachusetts readjusted their boundaries near Providence and Fall river. Eminent legal authority well versed in the laws and decisions regarding water rights assures me that the legal obstacles to Housatonic and Ten Mile water are such as can be readily overcome, providing the Connecticut Legislature will act in the neighborly spirit which we have a right to expect.

(*N*) Should negotiations for the Ten Mile river fail, there is a watershed lying north of the Croton, which is free from all interstate questions, and which can apparently be made to deliver 150 million gallons of water per day into the New Croton Reservoir, but does not have the advantage of elevation possessed by the Ten Mile and certain other sources.

A high-pressure supply from the watersheds adjoining the Croton on the north and northwest appears from my reconnaissance to be impracticable.

(*O*) An ample supply of excellent water up to even 1,000 million gallons per day, or so much of this as wanted, can be brought from the Adirondacks by gravity and delivered at the same elevation of 300 feet above sea level, at a cost, including all interest, depreciation, maintenance and sinking fund to pay off the whole cost in 40 years, for from $50 per million gallons to $25 per million gallons, according as a supply is taken of 200 million gallons per day, or of 800 million gallons per day, but while cost per million gallons is low, the total bond issue that would be required for this would be so extremely large as to make this project probably impracticable.

(*P*) By taking water from the Hudson river at Poughkeepsie and filtering it, and by constructing storage reservoirs in the Adirondacks, to let down water whenever, in extreme drought, there was any sign of brackishness in the Hudson water at Poughkeepsie, and then by filtering this Hudson water, just as Poughkeepsie has done for its own supply from the Hudson for 27 years, or as Albany is doing to-day, New York can obtain any desired quantity, even up to 800 million gallons per day, at a cost, including interest, maintenance, depreciation and sinking fund to pay off all costs of construction in 40 years, for water delivered at city limits at 300 feet above sea level, less than $50 per million gallons.

For filtered Hudson water delivered at city limits at same level as the Croton at Jerome Park, the cost will be about $7 per million gallons less than for water delivered at elevation 300.

(*Q*) If the Ten Mile and Upper Housatonic water can be secured, these sources will supply Brooklyn much more cheaply than water from Suffolk County.

If these sources are not obtainable, and if the New York Legislature will remove the restriction forbidding Brooklyn taking water from Suffolk County, then the surface waters and ground water from the saturated gravels above the blue clay in the region east of Massapequa can probably furnish from 75* to 125 million gallons per day of excellent water, at a cost a little less than $40 per million gallons.

***(A geological study of this watershed from Massapequa to Brookhaven raises serious question of it being so extensive as assumed in the Brooklyn investigation of 1896. On the other hand means can probably be devised that will probably yield a larger proportion of the total annual rainfall.)**

(*R*) A deep, artesian well supply from the deep gravels of Long Island lying beneath the blue clay, of any sufficient quantity to be of material value in solving the problem of Brooklyn's future supply, appears from a geological study, set forth in this report, Appendix No. 16, to be utterly hopeless.

(*S*) For Staten Island. A simple inspection of the ground or of the topographic map quickly shows that no adequate supply of surface water can be obtained on the island. A geological study shows the obtaining of any adequate future supply from subterranean strata to be utterly hopeless. The best means of supply appears to be by submerged pipes from Brooklyn, carried across a little north from the Narrows.

(*T*) West of the Hudson, the sources south of the Esopus are not very promising, and beyond all question are less advantageous than the Upper Housatonic.

The RAMAPO RIVER, which (with the Mahwah) would for $15,000,000, or somewhat more, furnish about 60 to 75 million gallons per day, probably cannot be diverted from flowing into New Jersey, because of its availability for the future supply of certain New Jersey cities. The Brooklyn report of 1896 appears to have underestimated the cost and overestimated the quantity of water available from this source.

The POPOLOPEN appears to be favorably located for furnishing about 20 million gallons per day of good water, at such elevation that it would flow by gravity, without pumping, to a level at the city limits of 300 feet above the sea.

The MOODNA apparently can be developed to furnish about 100 million gallons per day, at 300 feet elevation, but there is some question whether its water might not require filtration, or possibly prove unsatisfactory, even with filtration, because of peat and black muck in the bottom of some of the important reservoir sites.

The WALLKILL is attractive from its location on the map and because of lying at an elevation sufficient to deliver water by gravity at 300 feet above sea level at the city limits of New York. From its 417 square miles of watershed above New Hampton, by converting the Drowned Lands into an enormous storage basin of about 50 square miles area and about 25 feet depth, this watershed apparently could be developed so as to yield a safe flow of about 315 to 365* million gallons per day, if the estimate

* From personally observing the surprisingly small size of the stream, in comparison with that from other watersheds with which I am familiar, and from also briefly studying the flood marks, and from a hasty view of the geological conditions, I am led to question if the yield per square mile at the outlet of this basin would not be uncommonly small, because of percolation through deep strata, but I did not have time to investigate this matter of flow thoroughly.

A brief study of the geology of this dam site, which, from brief inspection taken in connection with the report of the State Geologist on Orange County, appears to be in a secondary valley eroded in a deep deposit of drift, and where formerly a dam of glacial drift converted the site of the Drowned Lands into a Lake, lends additional color to this suspicion of a small collectible yield, and without a year or more of gaugings of the actual stream flow, at some favorable spot, as near the railroad bridge at New Hamburg, I should hesitate to believe that nearly so great a yeild can be had from the Wallkill as that ordinarily obtainable from a watershed of this size with this proportion of storage. A thorough geological study directed toward this particular question is desirable, also a series of gaugings.

is made according to ordinary rules derived from experience on watersheds where there is an impervious substratum and an impervious dam at the outlet.

Six thousand acres, or 10 square miles, of the proposed reservoir bottom appears to be filled with swamp muck and peat deposits from 5 feet to 50 feet in depth, about 90 per cent. of which is organic matter, which, with shallow flowage under the summer sun would probably grow algæ as luxuriantly as they now grow onions, and there are doubts if its water could be satisfactorily purified and made potable by filtration.

There are important interstate flowage questions connected with flowing 10 or more square miles of land in New Jersey by this reservoir that would probably be almost as difficult to settle as any interstate question regarding the Housatonic.

Such examination as I have had opportunity to make on the Wallkill gives very small encouragement of this ever becoming an available source for the water supply of New York City.

The SHAWANGUNK RIVER would probably afford 35 million gallons per day of good water, at an elevation sufficient to flow by gravity to elevation 300 in New York, but at a high cost for storage and aqueduct, if the Wallkill storage basin proves impracticable.

Future Filtration.

(*U*) I venture to express the belief that by 20 years hence the public will have become educated to demand a higher standard of purity in public water supplies and that all future work should be laid out with a view to filtration 10 or 20 years hence of all water entering the distribution system. The conduits should run past land suitable for filter beds and head or fall be reserved suitable for working the filters without any expense for pumping. Personally, I believe that with complete meters and proper waste restriction filters could be properly advised at once for the Croton supply, and I find as detailed in the report that to filter the present supply with all its waste would cost only about 35 cents per capita per year.

Results of Certain Other Investigations Made.

Turning back now from consideration of possible new sources to the present water supply it may be added that:

A careful recomputation has been made of the actual run off of the Croton river day by day for the past 32 years, applying more accurate data to computing the flow from the old records of depth; whereby it plainly

appears that the computations heretofore relied upon gave too large a flow by about 10 per cent.

An investigation by experiments on full-size models of the Croton dam crests—made in the new hydraulic laboratory of Cornell University—proves that the formula used heretofore for computing the water wasted exaggerated this flow about 9 per cent.

A gauging of the flow in the New Aqueduct made at my request by the same assistant engineer, with the same current meter by which the New Aqueduct was originally gauged, proves that this has suffered an impairment of about 40 million gallons per day in carrying capacity, due probably to it not being kept clean.

The corrected results of the Croton river gaugings covering this period of 32 years, and of the Croton Watershed rainfall records have been condensed into tables and diagrams of great value for estimating the safe yield of such other watersheds as may be selected for new sources of supply.

The present rate of consumption of Croton water, in gallons per day, has been accurately measured with the result of showing that the estimates of consumption and of growth in consumption of water made by the Department of Water Supply at the time the Ramapo contract was urged were about 40 million gallons per day too large.

The rate of consumption of water in the Boroughs of Manhattan and Brooklyn at the various hours of the day and night has been measured with a degree of precision not heretofore attempted, thereby learning that in the quietest hour of the night or at 3 A. M. the draft in Manhattan is at the rate of 94 gallons per capita per day, and in Brooklyn, at the rate of 57 gallons per capita per day, thereby obtaining a better measure of the waste of water than has been heretofore available.

Experiments on the night consumption in the thoroughly metered cities of Fall River and Woonsocket have been made for comparison.

(*W*) A very full geological study of the deep subterranean strata of Long Island and of Staten Island has been made with a view to learning of the possibility of a supply from deep wells and of the best means of obtaining water from the saturated gravel. This, as already stated, shows that a deep artesian well supply, adequate for any general public supply is practically hopeless.

While the above facts and many others have been collected by the reconnaissances made by me in person and by others under my supervision, it is to be understood that *the shortness of the time and the broad field to be*

*covered has permitted only a work of reconnaissance on these surveys** and the estimates of cost may in some instances vary perhaps 20 per cent. from what would be given by a slow, careful, elaborate survey. Nevertheless, the figures presented are believed to be safe, for the intention has been to err on the safe side where data were doubtful.

In estimates of cost I have been guided, naturally, by the actual costs of similar work on a similarly large scale, carried on during the past 5 years for the water supply of Boston and vicinity by the Massachusetts Metropolitan Water Board, on which I was for a time one of the three Commissioners, and after allowing generously for the contingencies of incomplete surveys and increased prices of materials, I have in general assumed that it was possible for New York to secure equal economy.

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*The thorough investigation of water-supply problems is necessarily very expensive and takes much time.

The investigations leading up to the appointment of the Metropolitan Water Board to construct new works for Boston and vicinity, although carried on with the utmost economy, cost $42,500 and occupied a year and eight months, and, as a starting point, had data that had been quietly collected for years, had the benefit of some of the best expert advice to be had in this country given as a matter of public spirit free of charge, and were made by an engineer who had spent his whole professional life in that vicinity working on collateral problems, and who had for a long time previously been making studies in that direction and upon that particular problem.

The investigation made last year at Philadelphia upon improving the water supply is said to have cost about $30,000 and followed years of patient collecting of data and preliminary special study toward an additional supply.

The recent studies for improving the quality of the Pittsburgh supply occupied about two years and are said to have cost about $40,000.

The recent study for improving the quality of the Louisville supply by filtration covered nearly two years, and, although carried on under the very best of supervision, is said to have cost the city over $50,000, and to have cost the companies presenting proprietary devices to be tested probably as much more.

The studies for improving and extending the Cincinnati supply made within the past four years, including first, the study of sources, pumping stations and reservoir sites; and, secondly, a long series of filtration experiments, are said to have cost about $50,000. Preliminary investigations for the New Croton Aqueduct and for the New Croton Dam cost amounts, which are on record in your office, far in excess of any of the figures just mentioned.

I mention the above to illustrate the impossibility of covering this broad subject exhaustively in the time and with the means at my disposal, but believe that enough reliable information has been gathered for a safe and definite answer to your questions as above.

** To investigate the various subjects covered by your inquiry for the widely different conditions surrounding the different boroughs of Greater New York with the fullness of detail customary in large municipal work and with the completeness that I believe is desirable in the present case in order to clearly prove the best lines of development for the long future, would, with the fullest organization and most rapid work, wisely planned, take from one year to two years more time and the work of a dozen or twenty survey parties and the work of from three to six specialists in all, in chemical, biological and bacterial examinations, in filtration practice and in geology, in addition to the engineering experts, and, considering the breadth of the field to be covered and the varied requirements of the different boroughs, would cost at the least $100,000, or more properly $200,000, and under any but the most skillful direction, if thoroughly covered, would cost more. The least proper appropriation for such exhaustive examination as these problems deserve would, in my judgment, be $200,000, but surveys and investigations sufficient to justify beginning construction could probably be made for considerably less than $100,000 and probably within one year's time.

Recommendations for Action.

I respectfully present the following recommendations:

First—That surveys for an additional supply capable of development to a safe yield of at least 500 million gallons per day be authorized, and an appropriation of $100,000 made available for this purpose and with the expectation of an additional appropriation for extending and completing these surveys. That this survey be placed in charge of a committee of not exceeding 3 hydraulic engineers and carried forward in a broad, comprehensive spirit over the ground suggested on page 4, with special expert investigations of special problems and with all possible dispatch, and that a beginning be made as soon as possible.

Second—That the best of legal and other advice be at once sought as to proper procedure for obtaining secure rights to the diversion of the waters of the Upper Housatonic and Ten Mile rivers and that suitable application be made to the State of Connecticut regarding the diversion of these waters and regarding a change of boundary line by which the reservoir to be formed and the point of diversion would be within the State of New York and subject to its police control.

Third—That authority from the New York Legislature be sought for a supply to New York City from either the Upper Housatonic, Ten Mile, or from such other source considered elsewhere in this report as shall prove most expedient in case negotiations for the Upper Housatonic fail.

Fourth—That the prevention of waste of water by leakage from street mains and service pipes receive earnest, skilful attention by cutting the present pipe systems into sections by closing existing gates and measuring the night consumption in these sections from midnight to 4 A. M., through observations of small meters of the disc type set on a by-pass. Thereby discovering the most leaky portions of the distribution system and overhauling these, and that so far as practicable the methods of waste detection described on pages 67–69 be followed and remedy applied.

That the preparation of large scale maps be at once begun showing all taps and pipes and all connections, all obtainable information regarding size, age and condition of all these connections being systematically hunted down and plugged off entered thereupon.

Fifth—That new regulations for the setting of meters and service pipes be established. These to be in future put in by the city workmen at the city's expense, meters to be periodically tested and kept in repair by the city.

That improved, modern corporation cocks be used, that curb-stop cocks be systematiclly introduced at such a rate as will equip the entire city within 10 years.

That with a view to hunting down future leakage and electrolysis, multiple service branches from the mains, each supplying 2 or more houses, with an accessible corporation cock, be carefully considered and perhaps introduced along the principal streets for all future work; these subdividing to the several houses at or near the sidewalk line.

Sixth—That new regulations be so framed as to encourage the use of meters for all service pipes,* domestic, public, schools and hospitals, tug-boat and sprinkling cart supplies. That the draft of water-power from fire hydrants for street sprinkling or tug-boat supplies or any purpose except fire be forbidden.

That the statistics of metered water consumption be intelligently classified; first, according to use and again according to locality, by dividing the city into sections corresponding to the Venturi-meter subdivisions recommended on page 67, so that this use can be studied from year to year intelligently, and that it may serve as an aid to the detection of waste of water or under-registry.

Seventh—That proper steps be taken to learn accurately the desire of the voters, as a whole, relative to the complete introduction of meters, after first having brought about a better understanding of their economy and other advantages.

Eighth—That if after an earnest effort to restrict waste, enough water is not saved from replacing old and leaky pipes to materially improve the pressure downtown, one or more additional 48-inch mains should be laid along such line as appears most favorable after a careful study by self-recording pressure gauges set at various points to learn where the day pressure is most pulled down.

Ninth—That all of the reservoir dams be at once drilled for flashboard pins and that flashboards be set of the greatest height practicable as a means of augmenting the storage until the new Croton Reservoir is filled and a new

* The City perhaps paying for, or buying, with fair allowance for depreciation, all old and service-pipes to which meters are attached ?

supply secured. That the depletion of the reservoirs be carefully watched and regulated in proportion to the drainage area tributary to each and rigorous measures for checking consumption be applied if this depletion goes too far.

Tenth—That the effect of electrolysis on water pipes in causing leakage or corrosion of pipes be at once investigated, and this investigation steadily maintained from year to year and the necessary precautions to avoid serious injury determined and enforced.

Eleventh—That the depth gauge at head of New Croton Aqueduct be set to correctly indicate the depth; that the various locations at Croton Dam and along aqueduct and at all reservoirs where depths are regularly measured for estimating flow, be provided with good facilities for accurate measurement and that a higher standard of precision of measurement be insisted upon. That a new rain gauge be provided at Croton Dam and that two new rain gauges be established, one near Patterson and another in the southeastern quarter of the Croton Watershed.

Twelfth—That the New Croton Aqueduct be emptied at least for a day above Pocantico for inspection and cleaning.

Thirteenth—That a careful scientific study of the ground water supply available for Brooklyn be made along the lines suggested in pages 538 to 546, both with a view to extension beyond Massapequa and with a view to obtaining more water and better water from the present territory.

Fourteenth—That some arrangement be, if possible, made with the State Engineer and the Director of the United States Geological Survey by which topographic maps can be extended during the coming summer and fall over all the territory in which a water supply for New York can be reasonably sought.

Other recommendations will be found scattered through the reports under the various subjects treated which are better understood in connection with the accompanying descriptions, and therefore, need not be repeated here.

A Few Notes Upon Scope of Investigations and Degree of Completeness Attained.

Obviously the questions of first importance are—

What is the safe yield of the sources owned at present by the City?

What is the present consumption and rate of increase?

What proportion of present supply is wasted and can be saved?

Investigation of the first two questions will be found set forth in Appendices No. 1 and No. 2, so far as Manhattan and The Bronx are concerned. For Brooklyn the recent investigations of Mr. I. M. de Varona, Chief Engineer of the Brooklyn Water-works, made in 1895, give fairly complete answers for the conditions of to-day, and under the pressure of the other matters reported upon, time has not been found to put these questions of Brooklyn's consumption, waste and future requirements in such complete form as desirable.

Safe Yield:

To the first of the above questions—safe yield of present sources—a very large amount of time had to be given to get accurate results—much more time than was anticipated. For the former measurements were, as investigation progressed, found to have been made with a serious lack of precision. In various publications of the Department of Water Supply, of the Department of Public Works and of the Aqueduct Commission, statements appear implying that the safe yield of the Croton is anywhere from 250 to 500 million gallons per day.* No accurate complete analysis of the problem was found on record. In the recent active newspaper discussions widely varying views have been presented by men of high standing. Mr. A. Fteley, Chief Engineer to the Aqueduct Commissioners from 1890 to 1899, whose opinion is entitled to the greatest weight, informed me in conversation that he considered the safe yield not far from 265 million gallons per day, but had not had opportunity to analyze the questions so completely as he would have liked** (this new analysis shows 275 millions).

This question of the safe yield being at the very foundation of the problems presented, appeared to demand thorough treatment, and I present the results given in Appendix No. 1 with confidence that we now have this question of the safe yield of the Croton Watershed in such shape that it compares favorably with the best measurements of the kind made anywhere, and that

* Wegman's Water Supply states this as 280 millions.

**Mr. Fteley also showed me correspondence dating back a long time, in which he had strongly urged that authority be given his department to investigate the needs and possibilities for new sources of supply, he foreseeing long ago that the limit of the Croton supply would soon be reached.

the data available for estimating the Croton flow prior to 1900 have now been pretty nearly exhausted, and that this part of the problem can rest until the record of some more severe cycle of years of small rainfall appears in the future. The safe yield of the Bronx and Byram was also investigated, so far as the data permitted, with the result, as I believe, of showing that the estimate heretofore common (20 million gallons per day) is 33 per cent., or 5 million gallons per day, too high. This is set forth in Appendix No. 2.

Consumption:

The question of daily rate of total consumption of water in Manhattan and Bronx has also been gone into with a fair degree of thoroughness, and it is a relief to find that this is not so large and has not been increasing so fast as was supposed, but it is plainly increasing 15 million gallons per day per year.

Use and Waste:

The proportion of the water wasted in Manhattan and Brooklyn was investigated in a broad, general way by certain measurements of the comparative consumption by day and by night. This investigation of waste would have been extended to the other boroughs had time permitted.

The relative proportion of use and waste for Manhattan and Brooklyn is brought out so clearly by the curves of Diagrams No. 5 and No. 6 as to leave very little doubt that more than half and probably two-thirds of the total supply is wasted through leakage.

Costs:

On some other questions, as, for example, the cost of dams and aqueducts for an additional water supply, there has been no opportunity to go into the matter with great thoroughness, and my figures for the Ten Mile and Housatonic, although very carefully studied and based on careful reconnaissance in the field, are but approximations. Surveys more in detail may show here or there ten or perhaps twenty per cent. difference in the quantity of excavation or of masonry; perhaps in some places more, but more often less, for I have tried to be particularly liberal in anticipating the costs of the structures proposed, and of the damages liable to be claimed, for developing the Housatonic and Ten Mile supplies and have put in a very generous price for margin for contingencies. **I have figured costs throughout on the basis of well managed contract work at such labor prices as to-day prevail on the extensive constructions of the Massachusetts Metropolitan Water Board.** I trust that the margin of uncertainty in all these estimates due to lack of complete surveys, test-pits and borings, will be found so clearly stated that

none of my figures can seriously mislead or be misunderstood. For this purpose, also, and as an aid to a critical understanding by the non-expert, I have written into the estimates more notes than common of comparison with cost and yield of similar works, and made fuller note of the facts which I believe must lead others to substantially the same conclusions as stated.

New Sources:

A hasty reconnaissance was also made of all the possible sources of additional supply upon the map of a size worth considering for New York, which appeared to have any promise of future use. It is a matter of much regret to me that I have not had time to continue the explorations of the low level line northwest of the Croton Watershed by a line of accurate instrumental survey; and of further regret that I have not had time to go into details and make closer studies of the possibilities along the Moodna, Wallkill, Neversink and the Delaware (my investigation of the Delaware tributaries was very incomplete); but I have gone far enough on those studies to convince me that they are much less hopeful sources than the Ten Mile and Housatonic.

Memoranda and Explanations:

In the appendices will be found a part of the data that I have collected bearing on these various questions, some of which I have had to dig out laboriously from old reports, or by questioning old employees of the Water Department and by much personal inspection and work of assistants, and which it appears very important to leave recorded in such a way that it shall be conveniently accessible to others who may have occasion to investigate these matters, now or in the future. In these appendices I have also written up the basis of my estimates with more than ordinary fullness that they may be open to criticism and may be readily understood, and since it has appeared desirable that all the facts collected should be quickly available for the use of the engineers of the committee recently appointed by the Merchants' Association to investigate New York's water supply and open to their criticism and extension, advance sheets of the results of the computations upon yield and consumption and such of the other diagrams as appeared of interest, have been furnished them.

Coming now to the results of this work, my answers to your several questions are as follows:

***FIRST QUESTION.*—CAPACITY AND ADEQUACY OF THE PRESENT WATER-SOURCES.**

A careful recomputation from the daily measurements made by the aqueduct keepers during the past thirty-two years shows, as already stated, that although the consumption will very soon be uncomfortably close to the safe limit, with prudent management *the Croton Watershed, supplemented by the Bronx and Byram, and with a moderate and practicable restriction of waste, can furnish a safe and ample supply for the boroughs of Manhattan and The Bronx for four or five years to come; thus giving a short but adequate time in which to build works for an additional supply.*

This is shown in the most convincing manner by a study of Diagram No. 3-B in connection with Diagram No. 2, and will be found explained in detail in Appendix No. 1.

These recomputations and diagrams show that under special care, the Croton Watershed can be prudently relied on, after the New Croton Dam is finished in 1902, to furnish (taking some chances on impairment of quality of water), temporarily, say ..... 300 million gallons per day.

The greatest safe yield that can properly be relied on *permanently* after the new dam is completed and its reservoir filled, is ..... 275 million gallons per day.

Prior to the filling of the reservoir above the New Croton Dam, taking the storage reservoir capacity as we find it to-day, and *assuming the depletion of the several basins regulated with utmost skill* under a repetition of the most severe drought of the past thirty-two years, the safe yield would be..... 232 million gallons per day.

By means of adding flashboards on all dams to the greatest practicable height, temporarily, until the New Croton Dam is completed, and further increasing the storage by lease of ponds not owned, additional storage can be gained probably sufficient, with the rainfall the same as in 1880 and 1881, to make the safe yield prior to 1902..... 250 million gallons per day.

The actual average consumption of Croton water in the year 1899 averaged 226 million gallons per day.

The consumption has been increasing 15 million gallons per day each year for four years past, and four or five years hence will be greater than the Croton can be relied upon to furnish in a series of dry years.

The actual daily consumption as measured by the delivery of the New Croton Aqueduct to New York is found to be less than heretofore supposed by 38 million gallons per day, because of an impairment of the carrying capacity of the aqueduct, probably due to its not having been cleaned for nine years. The velocity of flow in an aqueduct is reduced when organic growths collect on its interior, just as the speed of a ship is reduced when its bottom is foul.

Former Estimates of Croton Yield 10 Per Cent. too Large.

The results of the recomputation of the yield of the Croton, day by day and month by month, for thirty-two years past, show the flow to have been less than the estimates previously published, by about 10 per cent. on the average, or 38 million gallons per day.

Thus, curiously it happens that the over-estimate of the Croton yield averages the same as the recent over-estimate of the consumption.

The cause of the excess of the earlier estimates of Croton river yield over the present estimates is fully discussed in Appendix No. 1, but may be briefly summarized here as follows:

The summary of the yield of the watershed is made up of four independent measurements:

1st. The quantity wasting over the Old Croton Dam.
2d. The quantity drawn through the Old Aqueduct.
3d. The quantity drawn through the New Aqueduct.
4th. The quantity taken out of the storage reservoirs to augment the flow, or, in turn, taken from the flow to refill the storage reservoirs.

The lack of precision in the old estimates came from:

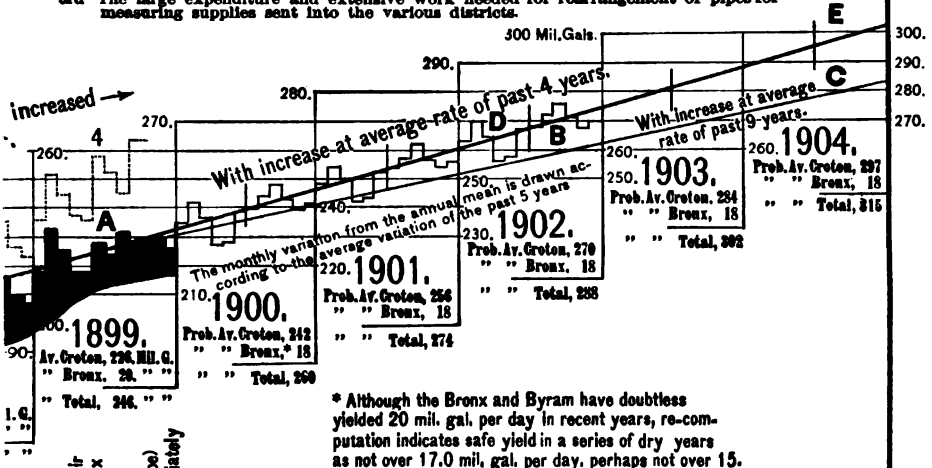
1st. A less accurate formula for computing waste over the dam than that which we now possess, and the use of only a single observation each day upon depth over dam—while there were two actually observed, both of which are now used.
2d. The use of assumed coefficients, which were too large, instead of actual gaugings for determining the flow in the Old Aqueduct.

There are strong reasons for expecting that consumption (and waste) will follow this upper line A - D - E unless :

1st Water meters are attached to substantially all service pipes.
2nd The systematic search for and replacing of old and leaky service pipes and mains is vigorously prosecuted.
3rd Distribution pipes are subdivided into numerous small districts with arrangements by Venturi meters, Deacon meters, or other appliances for measuring in bulk the total supply drawn into each district.

With the greatest progress that can be reasonably expected in prevention of waste the consumption will probably not be decreased below present consumption within these next 5 years, because of

1st The time needed to educate the ordinary house-holder that general introduction of meters is to his benefit.
2nd The time needed to establish systematic search for leaks and the great territory to be covered.
3rd The large expenditure and extensive work needed for rearrangement of pipes for measuring supplies sent into the various districts.



* Although the Bronx and Byram have doubtless yielded 20 mil. gal. per day in recent years, re-computation indicates safe yield in a series of dry years as not over 17.0 mil. gal. per day, perhaps not over 15. Therefore in projecting line into future a part of this deficiency is thrown on the Croton and 2 mil. gal. per day added to line.

Whenever City pipes are extended into region east of Bronx River to supply present deficiencies, an extra draft of 1 or 2 mil. gals. per day may be expected.

About this time, with completion of 2 new High Service pumps of 15 mil. gal daily capacity each, increasing present deficient pressure, the consumption will probably suddenly increase 15 mil gals. (per estimate of G. W. Birdsall)

Gates from Williamsbridge Reservoir and on some other mains from Bronx said to be throttled now to reduce pressure and check consumption; With these opened (as they should be) consumption would probably immediately increase 2 to 4 mil. gals. per day. (per judgment of Mr. Birdsall)

evival of Business

probable largely increased of water for manufacturing for business purposes

←—— Scant High Service Pumps, ——→

unable to keep up the pressure, reducing water level in houses 1 to 2 Stories and checking consumption.

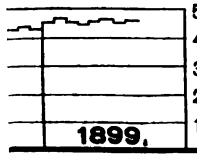


DIAGRAM No. 2

TOTAL CONSUMPTION OF CROTON WATER.

(INCLUDING WASTE)

In the CITY of NEW YORK-(Manhattan & Bronx)

Average Number of Million Gallons per 24 hour day in each month of the years 1884 to 1899, inclusive.

Based on recomputation of flow by J. R. Freeman C. E. Dec. 1899, from data furnished by G. W. Birdsall, Ch. Engr. Dep't Water Supply, by A. Fteley, Ch. Engr. Croton Aqueduct Commission, and from daily records of depths in Aqueducts by keepers at Croton Dam, Sing Sing, Tarrytown and Central Park, with tables of aqueduct flow corrected in accordance with gaugings by F. W. Watkins, Ass't. Engr. Aq. Com. on New Aqueduct Nov. 25-26 and Dec. 4-6, 1899, and Old Aqueduct Dec. 15-16, 1899.

also

PROBABLE CONSUMPTION for next 5 YEARS.

(as estimated by J. R. F.) if rate of increase is same as for the past.

3d. Lack of information which we now possess concerning impairment of carrying capacity of the New Aqueduct.

4th. A very incomplete allowance for the storage drawn and refilled.

The reasons for all these differences are now so firmly established and so fully set forth that any competent engineer who will carefully examine into the facts must be forced to depart from the earlier estimates of yield.

SAFE PERMANENT CROTON YIELD 275 MILLION GALLONS PER DAY.

Although after the enlarged Croton Lake is filled there will be available from the Croton Watershed and its storage reservoirs a supply of water, sufficient under careful management, by drawing everything to the limit, for the uniform daily draft of 300 million gallons per 24 hours in a drought as severe and long continued as any which has occurred during the past thirty-two years, this 300 million gallons per day is more than the greatest safe or proper rate draft to be relied upon permanently, for the following reasons:

1st. This continuous draft of 300 million gallons per day would draw the reservoirs down so far and so often that the quality of the water would be liable to impairment by the growth and decay of weeds on their margins, unless these were cut, raked and burned in the autumn at large expense. While in a great drought almost anything that is wet is welcome, and bad taste and bad color in water, resulting from long depletion of reservoirs, could be endured for a few months once in ten years much better than a water famine, I am led to believe that, with a view to freedom from bad tastes, etc., the Croton river works should be operated on a basis of not exceeding 275 million gallons per 24 hours.

2d. There is good reason to expect once or twice, or possibly three times in a century, a series of consecutive years of lower aggregate rainfall than in any period of three or five years, within this thirty-two years for which we have the Croton record. The Croton yield in the series of dry years, 1880, 1881, 1882 and 1883 was helped out in 1882, in comparison with certain other watersheds as shown by their rainfall records, by what appears to have been merely a "fluke," to use a popular phrase. In September, 1882, the average of the four rain gauges at Boyd's, Carmel, Middle Branch and Croton was 14.63 inches, or in this one month, and mainly in one storm, there fell one-third as much rain as often falls in an entire year. Various other eastern watersheds did not share in this good fortune and suffered much more severely.

3d. The capacity of the storage reservoirs has been figured as though all could receive their *pro rata* share of the run-off, while, in fact, several of the large reservoirs have relatively small water-sheds, and may not become half refilled before others begin to waste.* The New Croton Dam will largely remedy this defect, but the best of judgment in drawing the several reservoirs must be exercised even then to get out of each its full theoretic value; failing in this, the capacity will be smaller than 300 million gallons per day.

Comparison with Sudbury Yield.

Testing the figures of 275 million gallons per day adopted for the safe yield of the Croton from a study of its own record for thirty-two years by what has been found on another similar watershed, we find that the very accurate gaugings of the Sudbury river, forming a part of Boston's supply, give a rate of flow in the years 1879–1883, which, when applied to the Croton Watershed, with proper correction for differences of area of water surface exposed to evaporation, and volume of available storage, indicate that after the completion of the New Croton Dam, in a drought of equal severity and length to the Sudbury drought of 1879–1883, the Croton Watershed can supply 260 million gallons per day.

It is claimed that the Croton is a more productive watershed per square mile than the Sudbury. I have, therefore, made a new computation of the rainfall records (see end of Appendix No. 1), making use of the records from all of the rain gauges instead of merely the one rain gauge at Boyd's, and find that the long term rainfall records indicate that the average rainfall on the Croton is about 5.7 per cent. greater than the average rainfall on the Sudbury. Recomputing also the percentage of run-off to rainfall from the new data I find that for twenty-five years it averages 47 per cent. on the Croton Watershed against 49 per cent. on the Sudbury.

Adding, therefore, 5.7 per cent. for greater rainfall, and deducting 2 per cent. for smaller per cent. of run-off, we would expect 3.7 per cent. greater net yield on the Croton than on the Sudbury. Add 3.7 per cent. to the Sudbury figure of 260, and we get 270 million gallons per day safe yield for the Croton, after the Cornell Dam is filled. So while the Croton rainfall records

*On February 5, 1900, water wasted over the old Croton Dam at the rate of more than 3 billion gallons per day, while Amawalk Reservoir was still 50 feet below the flow line and the water levels in Boyds and Carmel each 20 feet below the top. In another heavy rain on February 13, 1900, water wasted over the Croton Dam at the rate of over 2 billion gallons per day, while Amawalk lacked 40 feet of being full and West Branch lacked 20 feet, and on March 17, 1900, although water has been wasting over Croton Dam steadily and profusely for six weeks, Amawalk still lacks 20 feet of being refilled.

are not perfect and the Croton rain gauges not well distributed, they serve to connect the Croton and Sudbury records fairly well, and to confirm by this independent means the reliability of the figure of 275 million gallons per day, already adopted in my computations, for the safe Croton yield.

Safe Yield Prior to Completion of New Croton Dam.

The quantity that can be safely relied upon the next three years, or until the new reservoir above the great dam is filled, is less than the 275 million gallons per day just stated, because of insufficient storage.

If the low rainfall of last summer and autumn is the beginning of a cycle of low rainfall, and if in the next two years, 1900 and 1901, we happen to have a repetition of the total amount of rain which fell in 1880 and 1881, then, taking all the present storage reservoirs on the Croton, including flowage rights on natural ponds, Mahopac, Kirk, Gleneida, etc., starting with them full on May 1, 1900, in addition to the natural flow of the river, every available gallon would be drawn out by the end of 1901, by a uniform draft of 232 million gallons per day.

If, in the effort to provide all practicable safeguards, temporary flash-boards, from 2 to 4 feet high, be at once placed on every dam, so far as practicable, thus gaining temporarily, say, 4 billion gallons of storage, as hereinafter described, and if, by the lease of right to draw ponds not controlled, and which were not drawn in the severe drought of 1891, but were drawn in 1880, there is gained a storage of 855 million gallons, and if the Central Park Reservoir be also reckoned as storage, then the reservoirs would be emptied at the end of 1901 by a uniform draft of 250 million gallons per day.

The safety of these two figures, 232 and 250, is also confirmed by an independent computation based upon the Sudbury records already referred to.

When estimating the increase of flow that can be had from the storage reservoirs the three following facts must be kept in mind:

1st. Some of the present storage reservoirs are located so far up toward the head waters that they do not quickly refill. To illustrate, in the recent heavy February rains, the water has been at times wasting over the Old Croton Dam at the rate of from 2 to 3 billion gallons per day, while certain of the large reservoirs were not yet half refilled (see foot-note on p. 24).

2d. There is the possibility of a more severe drought than that of 1880–1881.

3d. But, on the other hand, as a safeguard, there are nine chances out of ten that we shall not have so severe a drought as that of 1880 repeated within the next three years, and there is always present the opportunity to restrict waste if reservoirs are found to be becoming dangerously low.

We, therefore, appear safe in drawing from 232 to 250 million gallons per day prior to the filling of the New Croton Lake providing a close watch is kept on the rate of depletion and the downstream reservoirs always emptied first, and the upstream reservoirs lowered only in proportion to their watershed areas.

Over-estimate of Consumption.

While the safe yield of the Croton is smaller than expected, the present consumption is also found to have been over-estimated.

A measurement of the quantity conveyed by the New Croton Aqueduct made at my urgent request, upon November 22 and 23, 1899, and repeated, to make sure, on December 4 and December 6, 1899, by the same engineer (F. W. Watkins, C. E., Assistant Engineer, Croton Aqueduct Commissioners), who gauged this aqueduct in 1891, 1892 and 1893, using the same current meter (Mr. Fteley's), shows that there was an impairment of 38 million gallons in the carrying capacity of the New Croton Aqueduct existing in December, 1899, due mainly, in my opinion, to the aqueduct having been used nine years without cleaning. The result is that the consumption of water by the city has for a few years past been largely over-estimated and the rate of increase of consumption from year to year has been also over-estimated. Correcting these records carefully, month by month, I find that the consumption of Croton water during the past year (1899) has averaged 226 million gallons per day.

The rate of increase for the past eight years in Manhattan and The Bronx only, has averaged 12 million gallons per day.

The rate of increase for the past four years in Manhattan and The Bronx has averaged 15 million gallons per day.

Under present conditions, with the revival of manufacturing and commercial activity, an increase at even a greater rate than 15 million gallons per day for a few years to come is not outside the possibilities.

With only 14 million gallons per year increase added to consumption of 1899, we find the consumption of Croton water for a few years to come will be about as follows, if immediate and thorough means are not taken to stop waste—

MARGIN OF SAFETY IN CROTON SUPPLY.

Probable consumption of Croton water (exclusive of Bronx water) with same increase yearly as for a few years past. The Bronx and Byram sources being used to their full safe capacity meanwhile).	Yield of Croton water that can be safely relied on with present storage augmented by flashboards and with rainfall same as in 1880–1881. Million gallons per 24 hours	Surplus or shortage for rainfall like 1880 – 1881 with use and waste as at present.
For the year 1900.....242 million gallons per day.....	250	8 surplus.
" 1901.....256 "	250	6 shortage.
" 1902.....270 "	275	5 surplus.
" 1903.....284 "	300	16 "
" 1904.....297 "	300	3 "
" 1905.....311 "	300	11 shortage.

There is thus safe time to build new works, particularly if some of the present waste can be saved.

In the above table 300 is used, instead of 275, for safe yield, because some chance of impairment of quality of water by keeping reservoirs long drawn down could be tolerated in an emergency. *Our safety lies in the fair probability that the next two years will not be one of these abnormally dry periods, and in that, having knowledge of where we stand, the depletion of the reservoirs can be carefully watched and vigorous measures in throttling pressure and lessening waste applied if it goes too far.* A close watch and the best of management are needed, and with these secure, *the present supply is safe until a new one can be built.*

It would be an act beyond excuse for those entrusted with the management of the water supply of this city to again take such chances as, without knowledge of the public at large, were taken in the year 1891 by drawing upon the storage at such a rate as to leave it entirely exhausted on the 15th of November, with nothing between the city and a water famine, but a little water in the Central Park Reservoirs and the natural drainage of the stream, largely from the East Branch swamps. Providence was very kind in 1891 and sent rains just as the limit was reached. Had the fall rains been delayed in 1891 as in the past year, New York would have quickly found its possible

supply of water cut down for one or two months to the possible pumpage and drainage from reservoir beds and river margins, an amount surely not exceeding 50 million gallons per day.

To cut the water supply of a community in the year 1900 drawing 242 million gallons per day, and with pressure throttled demanding at least 200 million gallons per day, down to 50 million gallons per day, would bring danger from fire, stoppage of business and an amount of suffering and commercial disaster disquieting to contemplate.

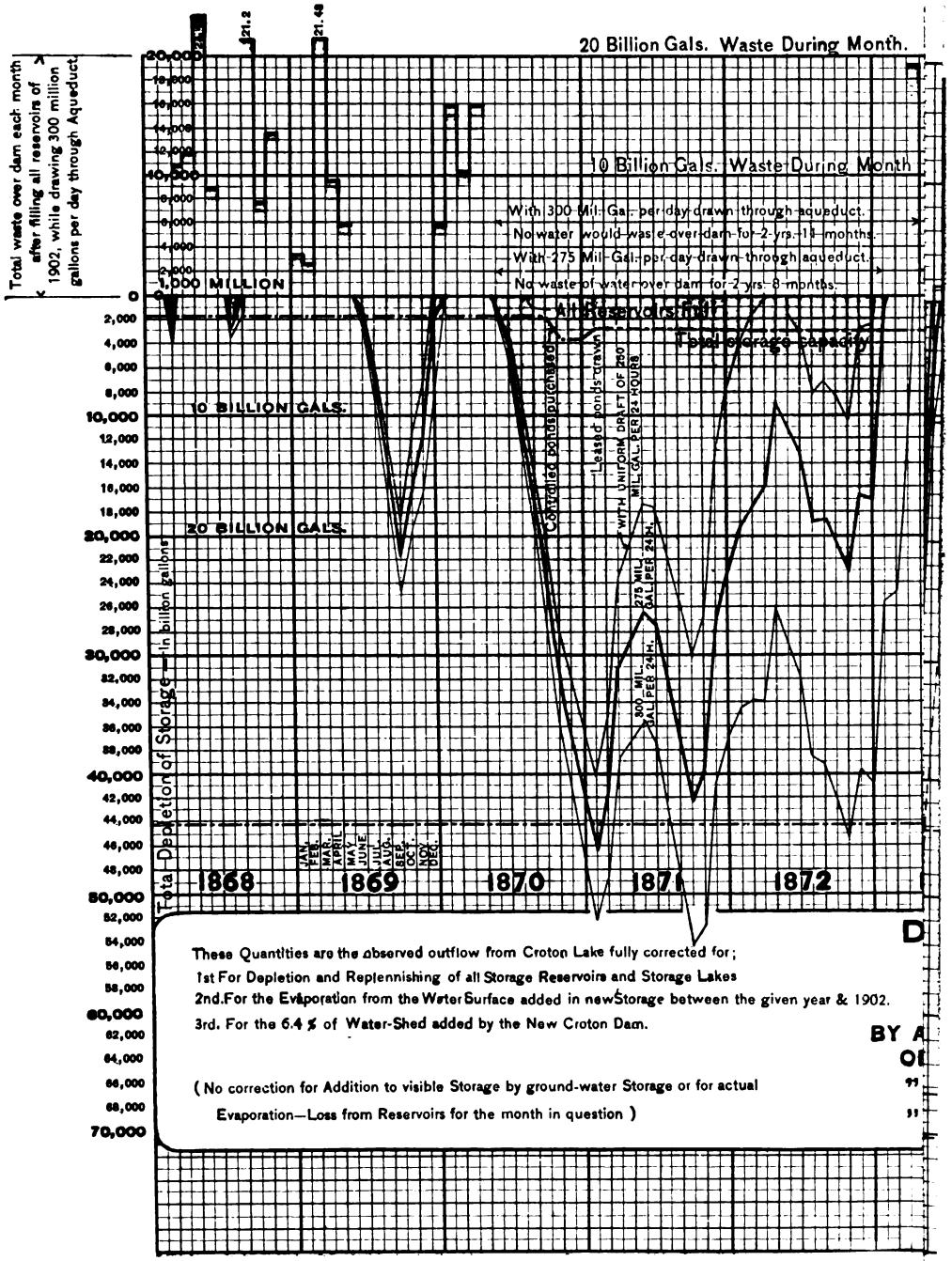
Detailed instrumental surveys for an additional supply ought to be begun at once, and a large number of field parties set at work and pushed ahead with all possible vigor, so that construction may, if possible, be begun early in the spring of 1901. The New Croton Dam will be at least 10 years in building; ground was broken September, 1892. The New Croton Aqueduct, after about 2 years had been spent in surveys, plans and securing land, was 5½ years more in building, from the time ground was broken until water was first run through it. These periods could be lessened by modern methods of excavation and by night work, under strong, vigorous administration, but at best it takes a long time to wisely locate and construct the dams and aqueducts of a great water supply.

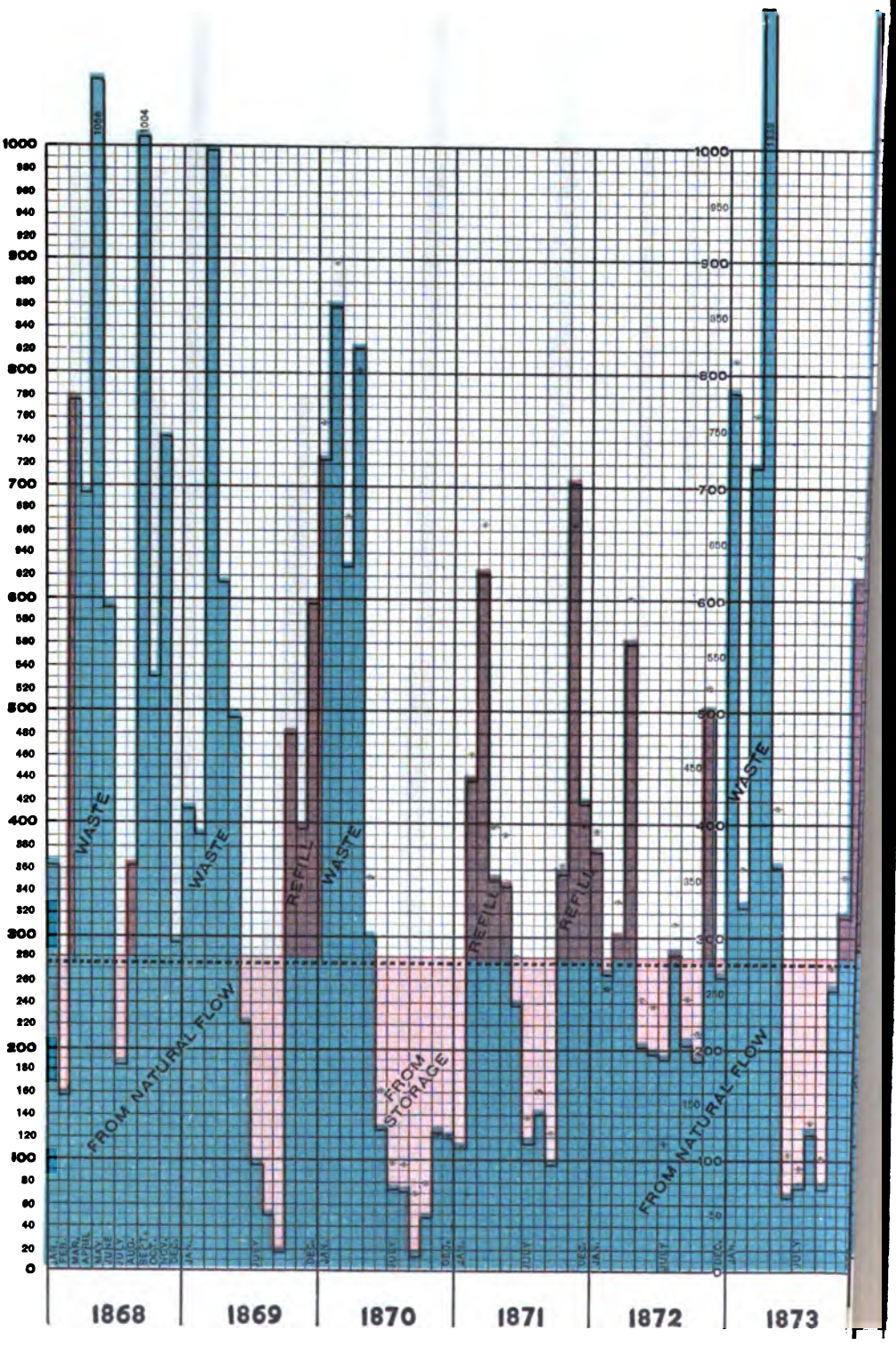
I do not believe that the hope of restricting waste justifies any delay whatever in making sure of an adequate additional supply; the time is scant at best. If there is to be delay in securing additional supply then a vigorous prevention of waste by introducing domestic meters must begin, or the Water Department and the citizens run for luck against the occurrence of dry years. It will take years to find all the bad leaks and they cannot be stopped at short notice.

Flashboard Storage.

Pending the completion of the New Croton Dam in 1902, in order to gain all possible storage as a safeguard against the chance of unusual drought in the next three to five years, I earnestly recommend that a careful examination of reservoir margins, dams and spillways be made at once with a view to the immediate drilling of all of the spillway crests for flashboard pins of soft steel rods from 1¼ to 1½ inches diameter, set 20 inches apart, and that 1-inch spruce flashboards, unplaned but with jointed edges, from 2 to 4 feet high, be placed upon every one of these reservoir spillways where this is practicable, with all possible dispatch.

If Mr. Birdsall's recollection of the circumstances affecting a temporary


20 Billion Gals. Waste During Month.
10 Billion Gals. Waste During Month
With 300 Mil. Gal. per day drawn through aqueduct.
No water would waste over dam for 2 yrs. 11 months.
With 275 Mil. Gal. per day drawn through aqueduct.
No waste of water over dam for 2 yrs. 8 months.
Total waste over dam each month after filling all reservoirs of 1902, while drawing 300 million gallons per day through Aqueduct.
1,000 MILLION
All Reservoirs full
Total storage capacity
10 BILLION GALS.
20 BILLION GALS.
Controlled ponds purchased
Leased ponds drawn
WITH UNIFORM DRAFT OF 250 MIL. GAL. PER 24 HOURS
275 MIL. GAL. PER 24 H.
300 MIL. GAL. PER 24 H.
Total Depletion of Storage — in billion gallons
1868
1869
1870
1871
1872
JAN. FEB. MAR. APRIL MAY. JUNE. JUL. AUG. SEP. OCT. NOV. DEC.
These Quantities are the observed outflow from Croton Lake fully corrected for;
1st For Depletion and Replennishing of all Storage Reservoirs and Storage Lakes
2nd. For the Evaporation from the Water Surface added in new Storage between the given year & 1902.
3rd. For the 6.4 % of Water-Shed added by the New Croton Dam.
(No correction for Addition to visible Storage by ground-water Storage or for actual
Evaporation—Loss from Reservoirs for the month in question)


1000
950
900
850
800
750
700
650
600
550
500
450
400
350
300
250
200
150
100
50
0
WASTE
FROM NATURAL FLOW
REFILL
FROM STORAGE
1868
1869
1870
1871
1872
1873

raising of the height of flow line are correct, which I have no reason to doubt, then it will be found possible by these flashboards to add temporarily about 5 billion gallons to the present storage, or about twice as much as the entire contents of the Boyd's Corner Reservoir, at an expense for construction that need not exceed five thousand dollars, in addition to the work of from 2 to 4 laborers to keep these flashboards "dusted down" and tight. If 5 billion gallons can thus be obtained, it will add nearly 10 million gallons per day to the city's water-resources until the New Croton Dam is finished. If investigation proves that all these flashboards to the full height estimated are inexpedient, even half of the above storage or only 2,500 million gallons is well worth securing at the full estimated cost.

Flashboards of this kind have been used many years on the dam 900 feet long across the Merrimack river at Lawrence, Mass. These are now carried there 4 feet in height, and in drought, either in winter or summer, are easily made tight by "dusting" them twice a day. They are safe, cheap and efficient, as I can testify from long personal experience in the service of that Water Power Company. The pins bend over instantly if the height of water becomes too great. As the flood subsides the bent pins are replaced by straight pins and the boards are replaced, without danger to the workmen.

A careful analysis of the present depletion of the reservoirs set forth on the Depletion Diagram No. 3 following, made about January 10, 1900, showed that notwithstanding that there was a greater depletion, or more total emptiness of storage on December 31, 1899, than has ever before existed on the Croton, this will, without doubt, all be filled except, perhaps, a part of Amawalk, and 5 billion gallons or more of water go to waste before the summer of 1900. These flashboards would save this waste water very cheaply. They are the cheapest safeguard that is in sight against a shortage of water, and I so advised Mr. Dalton, Chief of Department of Water Supply, orally, in November, 1899.

Increasing Croton Supply by Constructing More Storage Reservoirs.

A comparatively small increase of capacity for water supply can be given to the Croton watershed by building more storage reservoirs for the purpose of carrying water over several years, from a year of heavy rainfall to a year of drought, or, for example, storing the surplus of a year like 1878 and holding it idle ready for use in the extreme drought of a year like 1883, after several successive years of moderate and low rainfall.

I am led to believe that additional storage reservoirs in the Croton Basin are not now advisable and that with the completion of the great Croton Dam in 1902 or 1903, the development of storage on the Croton Watershed will have been carried as far as is at present reasonable or economical.

There is said to be a good site for building a dam at Cross river (dam site "N" of the Old Croton plans) to store about 9 billion gallons of water at a cost stated to me by Mr. Birdsall, as roughly estimated, of $1,500,000, for lands, structures and all attendant expenses. There are said to be one or two other possible sites (dam site "K-L" and dam site "O" of the Old Croton plans), each of nearly as great capacity, but of larger cost for development.

This addition by a dam at Cross river of 9 billion gallons of storage to the 65 or 70 billions available without flashboards in the year 1902, under the same rainfall conditions that prevailed on the Sudbury in 1880-3, would add about 12 million gallons per day to the estimate of 275 million gallons per day stated above, but to draw so much would add a year to the average period of depletion of the reservoir system as a whole and keep the reservoirs too long drawn down.

I have not had time to examine exhaustively into this question of a reservoir at Cross river, but from present information I am strongly inclined toward the belief that the limit of economical storage development for the Croton Watershed for the present will have been reached with the completion of the New Croton Dam in 1902, which will add somewhere between 23 and 27 billion gallons of available storage, making the total available storage 65 or 70 billion gallons, which is equivalent to from 189 million gallons of controlled storage per square mile of net land surface in the Croton watershed, to 203 million gallons of storage per square mile, the latter figure including every possible reservoir.

It is certain beyond question that all that could be gained from this proposed Cross River Reservoir would not supply the natural increase of consumption of water in Manhattan and The Bronx for more than one or two years. Work on a much larger and more comprehensive plan is needed, and more water for the money can be obtained outside the Croton Watershed as a part of a comprehensive plan than by expending a million dollars or more now at Cross river. Possibly in the distant future a reservoir at this site may become worth its cost; after filtration is available to remedy the ill effects of long depletion.

ADEQUACY OF BROOKLYN'S PRESENT WATER SOURCES.

For five years or more past the urgent need for an additional source of water supply has been so clearly apparent to all who have made themselves familiar with the Brooklyn Water-works that I have not thought it profitable to devote any considerable proportion of the brief time at my disposal to the making of measurements of consumption in Brooklyn or the compilation of further statistics regarding Brooklyn's needs. Five years ago this matter was ably investigated by Mr. I. M. de Varona, Engineer of the Brooklyn Water-works, under the direction of Mr. Alfred T. White, then Commissioner of City Works, and a report made to the Brooklyn Common Council, dated January 31, 1896. From the published copy of this report, pages 5, 26 and 27, we learn that Engineer de Varona concluded that the sources then in possession of the city, west of Massapequa, when fully developed, would have a safe yield in dry years of..... 114 million gallons per day,

but to collect and convey all of this to the city requires an aqueduct enlargement and other improvements not yet built.

The consumption from the public sources in Brooklyn in 1898, per report of the New York Department of Water Supply, pages 24–25, was about..... 94 million gallons per day,

with 10 million gallons daily in addition drawn from private water companies.

In 1899 the average consumption from public sources was 92 million gallons. Last summer the city was reported to be on the verge of a water

famine; this has helped to check the increase. So that for the last 3 months of 1899 consumption averaged 88 million gallons daily from public sources.

During the summer of 1899, Brooklyn came so near to the exhaustion of its visible supply that no further demonstration of the urgent requirement for additional supply appears necessary in this report.

The per capita consumption in Brooklyn is lower than in any other large city in this country, so there is no hope of increasing the available supply by decreasing the per capita consumption or decreasing the waste enough to more than take care of the new consumers coming in the period required for building the works for the new supply.

The Borough of Queens is in urgent need of additional supply. The conditions are in part set forth in pages 28 and 29 of the Annual Report of New York Department of Water Supply for 1898, but beyond the inadequacy of the present sources to give sufficient volume, it may be noted that the quality of the present sources, which are driven wells near to the territory already occupied by dwellings and factories and which will each year become more thickly populated, must become continually less desirable.

The Borough of Richmond, as is more fully set forth later in this report, has several considerable villages in which there is no water supply whatever and the present sources appear already taxed to the safe limit and appear liable to such pollution as the Island becomes more thickly populated, that they may ultimately have to be abandoned.

It would have required fully two months more time than that at my disposal to have collected the detailed information requisite for properly setting forth these matters in a statistical way, but the fact is apparent beyond doubt or question to any one who will give the matter brief study, that every one of the several Boroughs of Greater New York is in need of more water, due in most cases to the rapid growth in population.

TABLE NO. 1.

Consumption of Water per Inhabitant per Day in 40 of the Largest American Cities and a Few of the Largest Foreign Cities. Also growth in Population. Compiled by John R. Freeman, C. E., September to December, 1899.

NAME OF CITY. (American cities arranged in order of population.)	U. S. Census Population in 1890 to Nearest Thousand.	Per Cent. Increase in Population during Ten Years, 1880-90.	Local Estimate of Population in 1898 to Nearest Thousand.	Per Cent. Increase in Population for Ten Years, Based upon Increase from 1890 to 1898.	AVERAGE GALLONS USED PER INHABITANT PER DAY. Average for Five Year Periods of which Year Below is the Middle Year. 1875.	1880.	1885.	1890.	1895.	In 1898.	Per Cent. of Taps Metered in 1898.	Per Cent. of Consumption Metered in 1898.
New York	1,711,000	41.9	2,049,000	24.7	80	69	59	67	93	**111**	26.0	.....
Brooklyn	824,000	31.2	1,100,000	41.8	56	57	62	68	75	**88**	1.8	.....
Chicago	1,100,000	118.6	1,852,000	85.4	101	114	116	128	147	**148**	2.0	13.8
Philadelphia	1,047,000	23.6	1,400,000	42.5	60	69	78	125	167	**196**	.6	Less than 1.0%
Boston	528,000	27.5	650,000	28.9	73	87	77	83	103	**113**	5.6	.....
St. Louis	452,000	28.6	645,000	53.5	59	73	70	77	87	**88**	6.5	.....
Cincinnati	297,000	16.4	405,000	45.4	59	73	73	115	105	**95**	4.9	21.6
Cleveland	261,000	62.8	375,000	54.2	47	65	88	106	129	**138**	3.7	23.5
Buffalo	256,000	64.9	383,000	62.0	75	100	143	192	256	**333**	1 6	.....
New Orleans	242,000	12.1	264,000	11.4	...	...	...	...	40	**42**	....	.....
Pittsburg	171,000	46.6	218,000	34.4	...	...	172	210	215	**233**	....	Less than 1.0%
Washington	230,000	29.2	280,000	37.0	...	145	125	145	171	**171**	2.0	8.5
Montreal	217,000	....	299,000	63.0	69	61	62	66	71	**72**	1.7	12.1
Detroit	206,000	77.5	291,000	51.5	109	128	171	163	151	**135**	10.0	.....
Milwaukee	204,000	75.8	280,000	46.6	47	102	108	110	101	**83**	60.0	30.0
Toronto	185,000	146.8	200,000	10.1	50	62	91	79	95	**95**	5.7	.....
Newark	182,000	33 7	260,000	53.5	44	64	72	84	100	**106**	10.0	.....
Minneapolis	165,000	251.0	220,000	41.6	21	53	58	78	90	**97**	17.3	.....
Louisville	161,000	29.8	225.000	49.7	25	42	60	72	75	**70**	7.3	.....
Rochester	134,000	32.6	173,000	36.3	...	61	60	62	68	**72**	23.6	30.8
Kansas City	133,000	137.5	200,000	63.0	...	...	...	...	50	**52**	23.5	.....

Name of City. (American cities arranged in order of population.)	U. S. Census Population in 1890 to Nearest Thousand.	Per Cent. Increase in Population during Ten Years, 1880–90.	Local Estimate of Population in 1898 to Nearest Thousand.	Per Cent. Increase in Population for Ten Years, Based upon Increase from 1890 to 1898.	Average Gallons Used per Inhabitant per Day. Average for Five Year Periods of which Year Below is the Middle Year. 1875.	1880.	1885.	1890.	1895.	In 1898.	Per Cent of Taps Metered in 1898.	Per Cent. of Consumption Metered in 1898.
Providence	141,000	29.4	170,000	25.8	...	30	36	47	58	54	80.0	.....
Denver	107,000	197.4	167,000	70.0	...	...	...	95	158	174	1.0	5.0
Indianapolis	105,000	40.2	195,000	107.0	...	...	...	49	54	54	7.1	.....
Allegheny	105,000	32.9	138,000	39.2	...	...	168	189	223	239	....	Less than 1.0%
Albany	95,000	4.4	108,000	17.1	...	...	...	149	169	151	11.0	.....
Syracuse	88,000	69.2	125,000	52.6	...	...	...	...	65	76	45.3	48.7
Toledo	81,000	62.0	106,000	36.6	51	61	56	69	73	64	35.0	.....
Richmond	81,000	26.6	100,000	29.3	...	...	145	152	126	99	25.6	.....
Paterson	78,000	53.0	110,000	51.2	...	...	...	...	160	146	4.0	3.0
Lowell	78,000	32.2	87,000	14.4	28	38	54	89	80	77	47.0	25.0
Fall River	74,000	51.0	98,000	40.5	20	29	27	28	34	32	93.1	58.5
Cambridge	70,000	32.1	89,000	23.7	52	46	52	66	77	86	32.6	46.4
Atlanta	66,000	78.4	.....	....	...	40	43	35	43	....	99.0	.....
Wilmington	61,000	45.2	72,000	26.6	...	...	...	102	69	80	3.1	35.0
Lynn	56,000	47.4	68,000	26.5	35	33	42	47	64	70	17.0	15.0
Lawrence	45,000	15.4	55,000	27.8	...	43	59	66	57	60	71.2	35.8
New Bedford	41,000	51.9	58,000	51.9	...	...	81	91	92	103	8.1	14.6
Newton	24,000	43.5	30,000	22.1	...	26	32	40	60	59	80.0	55.0
Poughkeepsie	22,000	10.0	24,000	9.0	62	72	74	77	77	70	77.5	.....
Woonsocket	.....	....	27,000	....	..	...	...	...	...	28	85.8	.. ...
London	5,183,000	2.1	5,756,000	13.8	38	40	39	41	43	42	....	.....
Paris	2,425,000	....	.....	....	35	42	46	48	56	....	....	.....
Glasgow	618,000	....	724,000	21.5	47	50	51	50	52	53	....	.....
Zurich	337,000	....	407,000	25.9	...	41	45	55	53	....	....	.....

Notes relating to various items in Table No. 1.

New York—The population for Manhattan and Bronx by the Police census is used in 1890. This figure is about 190,000 larger than that given by the United States Census.

Brooklyn—The figures given for population and consumption are for the Ridgewood system of supply.

Boston—The figures given for population and consumption are for the Cochituate and Mystic systems of supply, which included also Somerville, Chelsea and Everett.

New Orleans—The conditions in New Orleans are abnormal, a smaller percentage of the population than is usual being connected to the city water works system. For instance, in 1895 there were only 4,800 taps, while in Atlanta, Georgia, a city with about the same consumption per inhabitant, but only with about one-fourth of the population, there were 6,266 taps.

Pittsburg—The figures given for population and consumption are for Wards 1 to 23, inclusive, which are the only ones supplied from the municipal water works system. The position of Pittsburg in the list is in the order of the total population.

Washington—Consumption figures given are from the Engineer Commissioner's report, which are based upon the population of the whole District of Columbia, of which about 6 per cent. is without water facilities. See "Eng. News," November 2, 1899.

Montreal—Figures in the 1898 columns are for the year 1897.

Toronto—The percentage of taps metered is given for the year 1895.

Kansas City—The average of the figures for consumption for 1896, 1897 and 1898 is given in the 1895 column.

Providence—The figures given for population and consumption are for the city and also the suburban territory supplied with water. The position of Providence in the list is in the order of the city population, without suburbs.

Denver—The consumption for 1887 is given in the 1885 column.

Allegheny—The average consumption for 1886 and 1887 is given in the 1885 column.

Syracuse—The average consumption for 1896 and 1897 is given in the 1895 column.

Toledo—The average consumption for 1876 and 1877 is given in the 1875 column. The percentage of taps metered is given for the year 1896.

Richmond—The average consumption for 1885, 1886 and 1887 is given in the 1885 column.

Paterson—The average consumption for 1895 and 1896 is given in the 1895 column.

Atlanta—The percentage of taps metered is given for the year 1895.

London—The population figures given are for the Water District.

SECOND QUESTION.—WASTE OF WATER AND ITS PREVENTION.

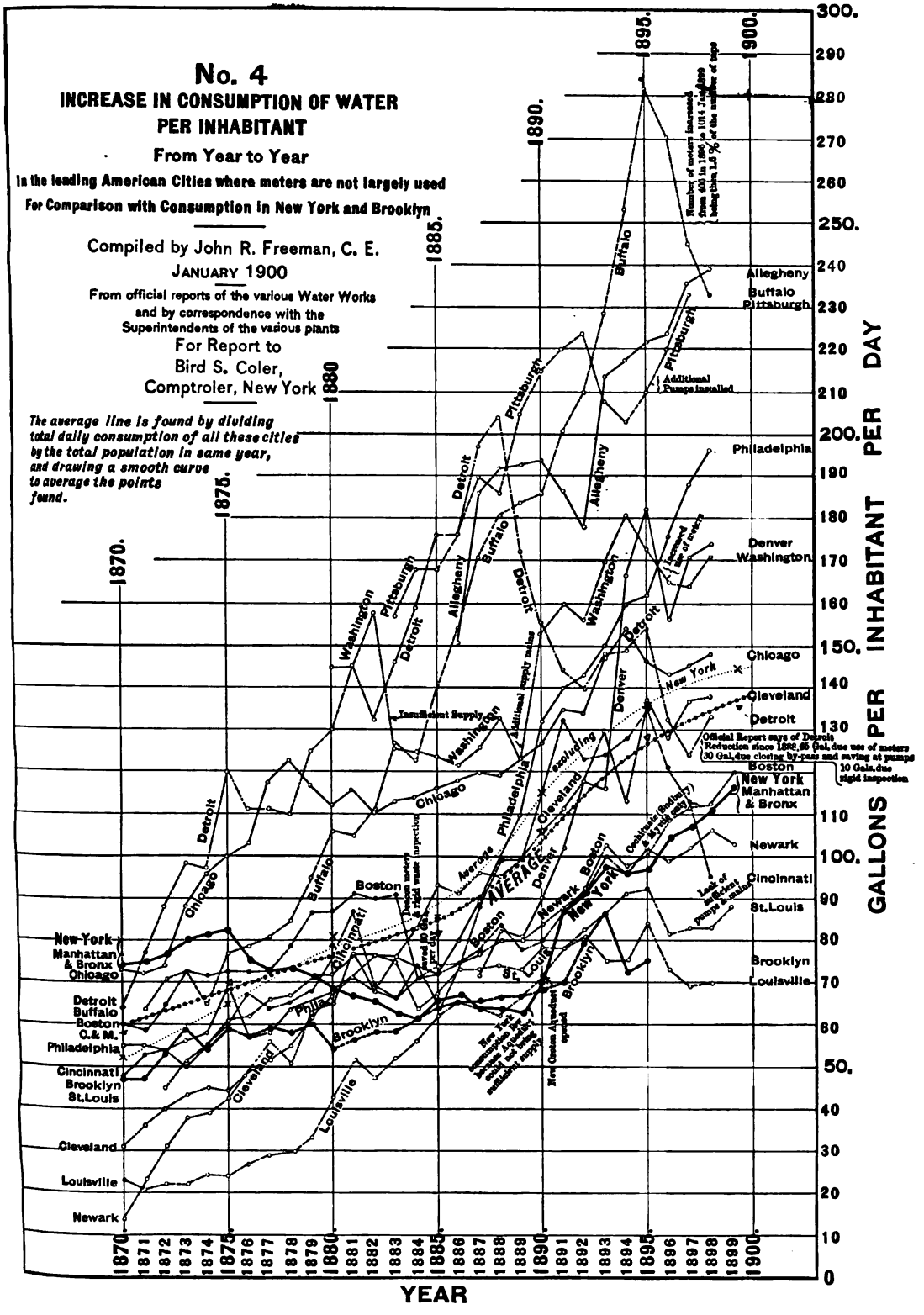
More than half the present supply is wasted. *The hydraulic and engineering features of preventing most of this waste are not difficult.* Meters for the whole of Manhattan would cost no more than has been stated to me as probable cost of the Cross River Reservoir and would, I believe, add twice or three times as much to the available supply. *The real difficulties on saving the waste, which I fear are at present insurmountable, rest in human nature,* and are questions of public sentiment, politics and good government.

The consumption and waste of water per inhabitant *is not greater in New York than in the other large American cities* (in fact, it is less), and I believe there has been a tendency in many recent discussions to exaggerate New York's rank in wastefulness of water and to over-estimate the saving that can be made, because of failure to appreciate that the real obstacles to reform are grounded in human nature, and not in hydraulic engineering; nevertheless, the waste is excessive in New York, just as it is excessive beyond reason in all of our other large American cities, and it is an engineering certainty that much of this waste, here or elsewhere, can be prevented if the average of public sentiment really so demands. Saving a part of this waste in New York and Brooklyn is the only means by which the possibility of a shortage of water in 1901 can be avoided.

Diagram No. 4* shows where the consumption and waste of water in New York and Brooklyn stands in relation to that of other cities. Table No. 1 gives data that could not be added to Diagram No. 4 without confusion, for all of the forty largest American cities which measure their total water delivery, and for certain large foreign cities. This was compiled from the answers to letters written to the Superintendents of various Water Departments, and in part from the printed water-works reports. It is particularly interesting in showing the tendency of all of our American cities toward increasing consumption along with the increase in conveniences of plumbing and increase in the number of tank ball-cocks, and to the increased use for manufacturing, steam, electric power, lighting, tall buildings, etc., and of this total increase, the experience of the metered cities in comparison with the unmetered, shows that with the more fixtures per house and the older pipes, *the waste is increasing more rapidly than the use.*

It will be seen from Diagram No. 4 that New York and Brooklyn each make a favorable showing compared with many other cities, and it is of

* A very similar compilation pointing to the same conclusion is to be found in the extremely interesting report of the Commission of Experts (Messrs. Hering, Wilson and Gray) upon the Philadelphia supply, made in October, 1899.


No. 4
INCREASE IN CONSUMPTION OF WATER PER INHABITANT
From Year to Year
In the leading American Cities where meters are not largely used
For Comparison with Consumption in New York and Brooklyn
Compiled by John R. Freeman, C. E.
JANUARY 1900
From official reports of the various Water Works and by correspondence with the Superintendents of the various plants
For Report to Bird S. Coler, Comptroler, New York
The average line is found by dividing total daily consumption of all these cities by the total population in same year, and drawing a smooth curve to average the points found.
GALLONS PER INHABITANT PER DAY
YEAR
AVERAGE
New York Manhattan & Bronx
Chicago
Detroit
Buffalo
Boston
C. & M.
Philadelphia
Cincinnati
Brooklyn
St. Louis
Cleveland
Louisville
Newark
Allegheny
Pittsburgh
Washington
Denver
Insufficient Supply
Additional supply mains
Increased use of meters
Additional Pumps installed
Official Report says of Detroit
Reduction since 1897 65 Gal. due use of meters
30 Gal. due closing by-pass and saving at pumps
10 Gals. due rigid inspection
Boston meters and rigid waste inspection saved 30 Gals. per day
New York consumption for Boston Aqueduct under construction
New Croton Aqueduct opened
Lack of sufficient Pumps & mains
Number of meters increased from 400 in 1895 to 1014 in 1899 being then 1.6 % of the number of taps

passing interest to note that the consumption of both Brooklyn and New York were quoted as comparatively low by the commission of eminent engineers that recently investigated the Philadelphia Water Supply, and in the report of the Engineer Commission which planned the new supply for Cincinnati in 1896, the consumption in New York was referred to as being "low because of its large use of meters." Stated in other words, while the waste of water in New York and Brooklyn is very large, the combined use and waste in New York and Brooklyn is not so large as in Philadelphia, Buffalo, Pittsburgh, Washington, Chicago, Cleveland, Albany, Detroit, Paterson, Boston, and a number of other cities.

By observations made under my supervision at the Central Park Reservoirs, described in Appendix No. 3 of this Report, it is shown to be probable that out of 116 gallons per inhabitant per day now regularly delivered from the Croton Aqueduct, two-thirds is wasted. In Brooklyn similar measurements show that one-half is wasted.

Without doubt this water wastes in New York, as elsewhere, through a combination of the following kinds of leakage:

Distribution of Leakage.

First, and probably much the most important is leaky house-plumbing.

Second, probably next, although possibly first in order of magnitude, old and leaky service pipes between the street main and the cellar, and abandoned service pipes.

Third, probably next in magnitude, is loss from the leaving of water-cocks unnecessarily open by the careless 10 per cent. or 20 per cent. of the householders.

Fourth, a smaller proportion, through leaky joints in New York's 833 miles of distribution mains, and occasional rust holes or cracks in some of the pipes laid 50 years ago that were cast on the side, according to the defective methods of early days and in pipes near salt water.

Fifth, there is a possibility that the embankments and beds of the distributing reservoirs are not absolutely impervious.

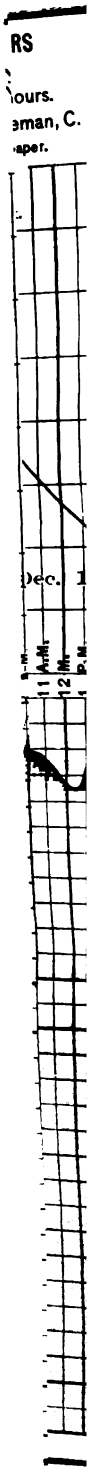
Sixth, we must not ignore the fact that the stealing or "unlawful diversion" of water through surreptitious connections has happened in other cities, since the time of Frontinus in Rome, and is said to have happened in New York, and that this would be very difficult of discovery under present conditions, but the percentage of the whole waste or loss that can be taken through surreptitious connections here in New York, must, I believe, be very small.

Waste *vs.* Use.

The relative proportions of waste and use in New York and Brooklyn were measured by means described in Appendix No. 3.

The basis of the method followed in these measurements is the well-known fact that about nine-tenths of the real use of water occurs in the hours of activity, or between 5 o'clock in the morning and 12 o'clock at night; while the waste of water through leaks goes on at pretty much the same rate throughout the 24 hours, night and day. Therefore by measuring the rate of draft going on in every hour of the 24, the rate at which water disappears from aqueduct and reservoir in the quietest hour of the night—a little after midnight, after the house tanks have become filled, or say from 2 to 4 A. M.—*is mainly a measure of the waste* while the excess of rate of draft in the hours of activity over this minimum night rate is an approximate measure of the actual use.

Diagram No. 5 gives, in the lower half of the sheet, these hourly variations in the consumption in Manhattan, and merits careful study. It is interesting to follow this rate of consumption through from hour to hour and from day to day, to note the larger use on Monday, the small use on Sunday, the maximum point at 10 A. M., the gradual tapering off at night as tanks refill, etc. The area on the diagram colored blue is taken as representing the portion really used, and the area colored red, the portion wasted. These observations were made by two sets of observers working in 12-hour shifts, each man taking observations once every six minutes for the entire twenty-four hours. The observations continued ten days, were made with much precision of measurement, the assumptions involved in the computation all appear reasonable, the chances for error were carefully reviewed and appear to have been effectually guarded against, and the same large consumption of water is repeated night after night and day after day with remarkable agreement. While I feel that one should go slowly in drawing conclusions from any single series of observations, nevertheless, after a study of the best available maps of the pipe distribution system and inquiry into the methods in use in its regulation, and after having reviewed the methods of my measurement under both friendly and adverse criticism, no good reason against their accuracy has appeared, and I can see no good reason for distrusting the substantial accuracy of these curves of rate of draft in Diagram Nos. 5 and 6. The curves of Diagram No. 5 apply to the entire supply from the Croton. It will be noted that even in the early hours of Sunday morning after the house tanks and the factory tanks have all had ample time to become refilled long before most citizens



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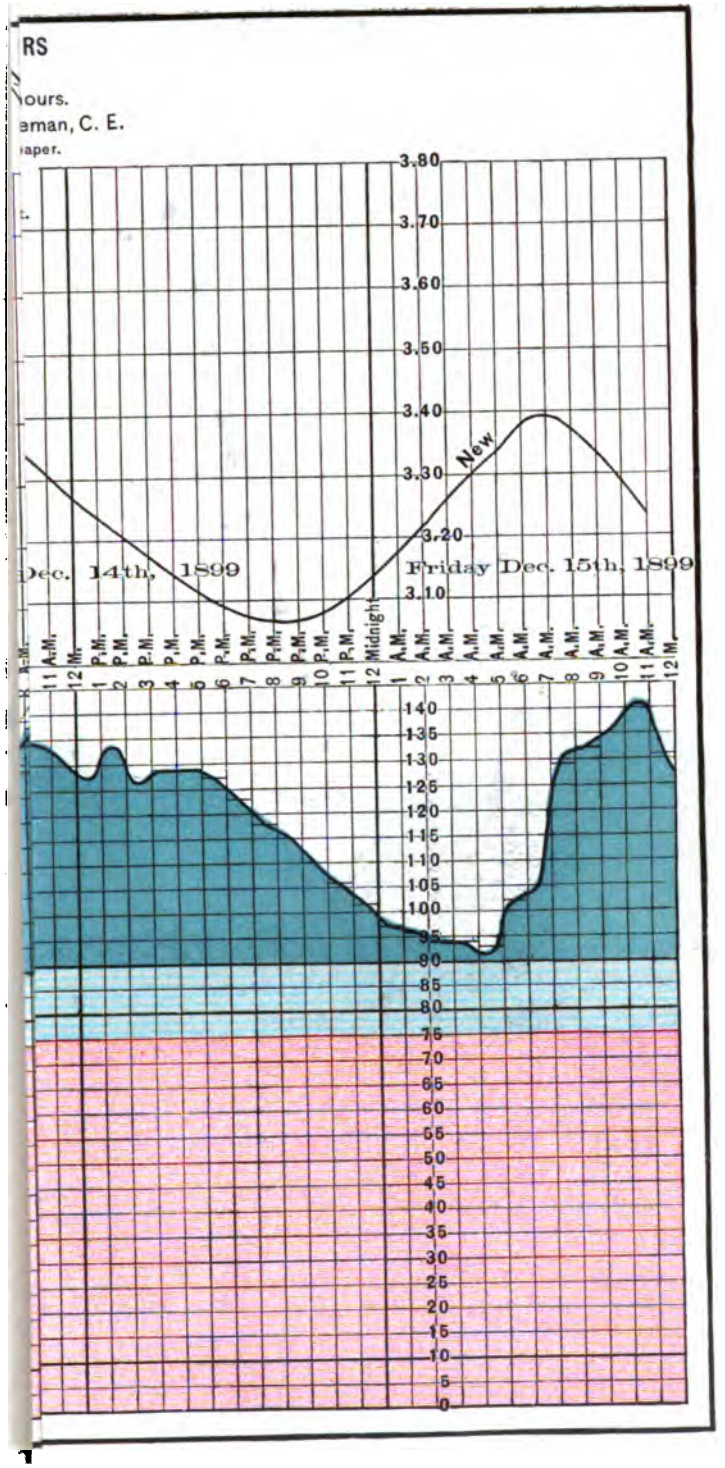

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are awake, and *at an hour when the real use must have been almost zero, the draft or outflow from the pipes did not fall below the rate of 94* gallons per inhabitant per 24 hours, but in order to give ample allowance for the New York consumption by night being greater than is most cities, I have called the rate of night use 19 gallons, which makes the waste 75 gallons per inhabitant per day. This leaves for the average rate of use at the busiest hour of 10 A. M., on Monday morning, 71 gallons per inhabitant per day, and indicates that *the average 24-hour 7-day use throughout Manhattan is only about 40 gallons per inhabitant per day.*

Confirmation of Tests of Leakage.

After the preparation of Diagram No. 5 and the presentation of copies to various engineers for criticism and suggestion because of the startling proportion of waste which it revealed, some suggestions were received among which the following demand special consideration:

First—It was asked if the accuracy of measurement might not be impaired by the fact that there were by-pass connections open around the Central Park Reservoirs, from the end of the Croton Aqueduct at One Hundred and Thirty-fifth street directly into the distributing mains, so that all the water used on Manhattan Island did not pass through these Central Park Reservoirs. In reply, a careful consideration will show that this would not affect the accuracy of the measurement. The logical form of the demonstration is—

1st. Substantially all the water consumed, used and wasted on Manhattan Island came from the New Croton Aqueduct.

2d. The rate of delivery of this aqueduct was maintained almost absolutely uniform, day and night, for two weeks or more.

3d. The only storage place of noteworthy size* into which a surplus delivery of the aqueduct could be discharged in those hours when rate of consumption was smaller than the uniform aqueduct delivery, was the Central Park Reservoirs, and those Central Park Reservoirs are the only storage from which the reserve could be taken in those hours of greatest activity when rate of draft of water is larger than the uniform rate of aqueduct delivery.

4th. Therefore the rise and fall of the Central Park Reservoirs must be a reasonably accurate measure of the hourly variation in rate of consumption of Croton water.

5th. Therefore, by deducting the reservoir fill by night from the uniform Aqueduct delivery, or by adding the reservoir depletion by day to the uniform aqueduct delivery, an accurate measure of the rate of consumption at any hour of the twenty-four can be obtained, irrespective of any change in the gates, and regardless of whether a part of the water or the whole of the water is by-passed into distribution, so long as there is a fair-sized open connection between the reservoir, the Aqueduct and the distribution system.

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* The high service reservoir is so small (one acre) as compared with the 125 acres in the Central Park Reservoirs that its modifying effect can be disregarded. Its fluctuations were observed through one 24 hours and found to affect the per capita result by less than 0.25 gallons. Any error in the unaccountably high night delivery of the high-service pumps can at most affect the final result for the whole city by only one gallon per capita. Computation indicates that any possible open connection between High Service and the Bronx would not affect the result for the whole city by more than one-half gallon per capita per 24 hours. Observations on high-service pumping were rendered incomplete by an interference elsewhere referred to.

In the present discussion, water drawn to refill house tanks is classed with consumption.

To be conservative, and to avoid possibility of over-estimating the waste, I have revised my earlier estimate of 80 gallons per capita by throwing all the margin of uncertainty in measurement, so far as known, in the direction of increasing the use and diminishing the waste.

This question of the proportion of use to waste is so important and it is so difficult to secure facts instead of opinions, or to secure figures that may be safely applied to the city as a whole from tests on isolated houses, that I also sought information as to difference in day and night consumption in Brooklyn and was courteously afforded facilities for investigation by Mr. de Varona. The method of measurement followed is described in the latter part of Appendix No. 3 and the results given in Figure 54, also in Figure 5. I

Rate of Night Use—A second criticism or suggestion was that perhaps there was a larger use of water in the quietest hours of the night, and it was suggested that it might be the custom of engineers in large commercial and industrial buildings to fill their tanks after midnight, when the city water pressure was higher and the work easier.

In reply:

First—The rate of 19 gallons per inhabitant per 24 hours, which I have allowed for the actual use at night, already includes a very liberal allowance for the peculiar conditions affecting night use in New York.

Second—I find that it is *not* the general custom of night stokers or superintendents of large buildings to pump any considerable part of the day's supply from the city mains after midnight, and in the principal districts downtown the water pressure now stands only 10 feet higher by night than by day, as shown by a few night records from a recording guage at a representative location.

To make sure of this, I had Mr. Harry P. Barr, mechanical engineer, of No. 141 Broadway, New York City, a gentleman familiar with the city and having acquaintances among the superintending engineers of large buildings, spend about 10 days in diligent inquiry about night use among the superintendents, engineers and night firemen of a large number of typical establishments, commercial and industrial buildings, power stations, steamship piers, railroad stations and hotels. The general result of this inquiry was to prove conclusively that the proportion of water taken from the city pipes between midnight and 5 A. M. by the above-mentioned typical classes of large users is extremely small.

There is more evidence of a late draft in refilling tanks in the high-service district than in the low-service or downtown districts. Downtown the uniformity of water pressure has been much improved within a very few years past by additional large pipes and by opening the gates wider, so that now the water level fluctuates only about 10 feet from night to day for an average of the important localities; while uptown, in the high-service district, the conditions have been steadily growing worse as pump capacity has become outgrown. Within about a year the water level by day is said to have fallen about one story in height and to be now 20 or 30 feet lower by day than by night.

Judging by the rate of fall of the night curve on Diagram No. 4 soon after midnight on Sunday, December 3, and again on Sunday, December 10, the tanks had become filled and the night flow constant by 2 A. M. On comparing the night curves of the different cities given in Diagram No. 5, the larger draft in the early night in New York stands out plainly, but it appears that we throw all uncertainty in favor of use and surely avoid an over-estimate of waste when we call the minimum rate of use at night 19 gallons per capita.

The night use finally adopted in my computations at rate of..... 19 gallons per inhabitant per 24 hours

is a much larger rate of night *use* than that found in the few cities where there are means for measuring the *use* apart from the *waste*, and where the rate of combined use and waste have also been measured.

For example, as shown by Diagram No. 6,

Fall River minimum consumption is at the rate of..... 12 gallons per inhabitant per 24 hours.

Woonsocket minimum consumption is at the rate of..... 9.5 gallons per inhabitant per 24 hours,

and deducting from these a probable *waste* of 6 or 7 gallons, leaves only about 4 gallons per inhabitant as probable for the rate of *use apart from waste* at night from 2 to 4 A. M. in these cities against the 19-gallon rate assumed herein to exist in New York.

also sought information as to the proportion of use to waste in those cities in which, so far as I can learn, the greatest progress has been in the prevention of waste; namely, Fall River, Mass., and Woonsocket, R. I.

Each of these is a compactly built manufacturing city in which the rate of domestic consumption would be a safer guide for the present purpose than that of certain suburban communities like Newton, Mass., and some others in which water meters have been largely introduced and water waste strenuously repressed. Both in Fall River and in Woonsocket it happens that there is no abnormal use for the very large factories or for processes requiring unusual volumes, for in each city nearly all of the factories that make exceptional use of water are alongside of a river or pond, from which the factory supply is taken, and in each of these cities the distributing reservoir for equalizing hourly fluctuations happens to be a tight steel tank, thus eliminating the question of leaky reservoirs. Fall River, in particular, is so large and with such varied interests and such a variety of residences, that its curve of rate of consumption and its total consumption are of special interest.

The record of metered use in larger commercial centres, like Boston or Chicago, would, of course, be a more satisfactory guide, but in the cities of this class unfortunately no good data can be found, for in them waste restriction has made no more substantial progress than in New York.

In Fall River and in Woonsocket, observations of the hourly variation in consumption were made, at my request, night and day for a week. Mr. P. Kieran, Superintendent of the Fall River Works, and Mr. Byron W. Cook, Superintendent of the Woonsocket Water Works, extended every possible courtesy in this matter, and the observations were made by competent observers with much care.

Each city pumps its supply into large steel tanks and in each case the troublous question of reservoir leakage is thus eliminated. At Fall River two of the three-million-gallon steel tanks were shut off, throwing all variation in level onto the third, which was regularly observed. The pump counter and length of stroke were also regularly observed. The pump slip was assumed to be 4 per cent., from a weir determination made some months ago. In Woonsocket the pump does not run at night and the change of level in the tanks is recorded automatically by a Winslow recording depth gauge.

The observations for the successive days nearly all repeat very well and the entirely independent determinations in these two different, but thoroughly metered cities, agree very remarkably.

That the average habits in hours of washing and hours of work and rest and in extra volume of water required by day per capita for the thousands and the millions of persons located in Woonsocket, Fall River, Brooklyn and New York average pretty much alike when the community is taken as a whole, is shown by the wonderful agreement of the curves of Figure 5.

Figure 6 indicates that *the difference between the per capita consumption of New York and Brooklyn and that of Fall River and Woonsocket is mainly a question of restriction of waste.*

Summarizing the results of these observations on the Central Park Reservoirs, we find the following:

PROBABLE PROPORTION OF USE AND WASTE OF CROTON WATER.

Estimated population supplied by Croton only—total for high and low service..... 1,947,000

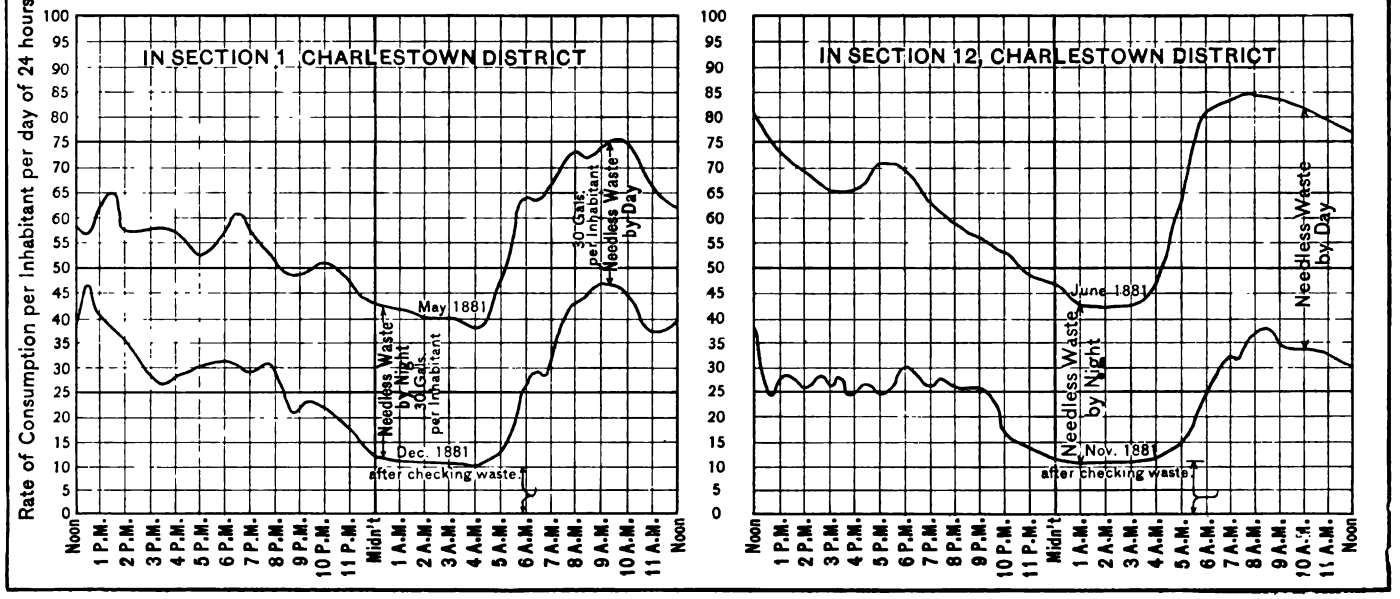
OBSERVATIONS MADE DECEMBER 2 TO DECEMBER 5, 1899, DECEMBER 8 TO DECEMBER 15, 1899, IN VERY MILD WEATHER, WITH NO ICE AND NO OCCASION FOR WASTE TO PREVENT FREEZING.	GALLONS PER INHABITANT, PER DAY.		MILLION GALLONS PER 24 HOURS, CROTON ONLY.	
Probable average amount of water really used.....	....	40	....	78
Probable average amount of water wasted.....	75	....	146	....
Assumed incurable waste	....	10	....	20
Needless or curable waste (probably).....	....	65	....	126
Total uniform rate of delivery by Croton Aqueduct		115		224

No. 8

HOURLY VARIATION IN THE CONSUMPTION OF WATER

Observed in certain districts in Boston, Mass.
In May and Dec, June and Nov., 1881.
Under supervision of Dexter Bracket. C. E.
with Deacon meter.
BEFORE and AFTER a rigorous investigation and checking of waste from main pipes, service pipes, and house fixtures.

For Comparison with Results of observations on New York's Consumption by
John R. Freeman C. E.
February, 1900.
Geo. A. Taber, Del.


Rate of Consumption per Inhabitant per day of 24 hours
IN SECTION 1 CHARLESTOWN DISTRICT
May 1881
Needless Waste by Night 30 Gals. per Inhabitant
Dec. 1881
after checking waste.
30 Gals. per Inhabitant Needless Waste by Day
IN SECTION 12, CHARLESTOWN DISTRICT
June 1881
Needless Waste by Night
Nov. 1881
after checking waste.
Needless Waste by Day
Noon 1 P.M. 2 P.M. 3 P.M. 4 P.M. 5 P.M. 6 P.M. 7 P.M. 8 P.M. 9 P.M. 10 P.M. 11 P.M. Midn't 1 A.M. 2 A.M. 3 A.M. 4 A.M. 5 A.M. 6 A.M. 7 A.M. 8 A.M. 9 A.M. 10 A.M. 11 A.M. Noon
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75 80 85 90 95 100

The proportion of waste in New York and Brooklyn revealed by these measurements is so startling that it is sure to be criticised and doubted at first; I have, therefore, as further confirmation of the method and result, transcribed some similar curves obtained from certain small districts in Boston before and after a season's strenuous work in discovering and stopping leaks. These are given in Diagram No. 6 immediately following. The saving in these two districts of Boston effected by the season's work of elaborate investigation, inspection and repair amounted to 30 gallons per inhabitant per day, and for the whole of Boston proper, or more exactly for the portion supplied from the Sudbury and Cochituate works, the application of this thorough system of inspection saved 29 gallons per inhabitant per day during the five months, April to August, inclusive, of 1884, as compared with the same months of the previous year.*

Requirements for Preventing Waste.

Returning again to the subject of this great waste of 70 or 80 gallons per inhabitant per day in New York City, arising from the five causes mentioned on page 37. To stop this waste, we must first find where it is, and to locate it, we need—

1st. An accurate daily record of the water delivered into the Central Park Reservoirs and the city distribution mains from Croton and Bronx. This we already have, through the aqueduct gauging, but greater precision of measurement is needed in future.

2d. A meter on every service pipe, whether it supplies a factory, a store, an office building, an apartment house, a dwelling or a city building; also a metering of tow-boat supplies and every kind of special use. This is by far the best remedy for leaky plumbing and careless or willful waste.

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* Lest one be misled by this high proportion of waste to use in New York and by the hope of checking it, into advocating too small a future supply, we must not leave this subject of waste reduction in Boston by Deacon meters and close inspection without also quoting from the experience of Boston to illustrate the difficulty or impracticability of a complete and permanent cure for waste throughout a large city by this method. Although eighteen years ago Boston had established what was probably the most complete water waste detection service that existed in any American city (other than complete metering), having about 80 Deacon meters in use, and although the consumption of water in the city as a whole *for the entire year* was lessened about 23.5 gallons per inhabitant per day, or from 91.5 gallons per capita in 1883 to 68 gallons in 1884, by this method combined with thorough inspection of house fixtures, but as its efficiency has since been relaxed, and the method has lost its novelty, the consumption has increased, until in 1899 it averaged 110 gallons per inhabitant per day, or about the same as in New York.

3d. By subtracting the water thus bought and paid for from the gross amount delivered by the aqueducts, it would be learned for the first time just how much was wasted in New York *outside the houses*, and I have little doubt this discrepancy between the aqueduct flow and the sum total of the meter readings would be found, as matters stand to-day, to be about 50 million gallons per day wasted in leaks outside the cellar walls in Manhattan and Bronx alone.

4th. To locate these leaks in an efficient and practical way, we must subdivide the distribution system; first, into districts of, say, at most, half a mile square, each of which should be fed by not more than 2, or at most 3, connections, and on each of these Venturi meter should be placed, measuring every gallon that goes into this district, and thus by subtracting the service-meter records from this main Venturi-meter record, the amount of street-main-waste and service-pipe-waste in each district would be measured.

5th. Next, these districts of about half a mile square should progressively be examined, taking the worst first, to locate and check the individual leaks. Doubtless the best method for this would be a modification of the Deacon meter system. Temporarily subdividing the district into sections each of about 1,000 feet square by closing all the water gates around its boundary that lead into it, except one, and on this one place a Deacon registering meter that could be moved from one section to another within the district as the examination progressed.

The mere presence of water meters on the service pipes of all consumers would, according to my best guess, save at least 50 million gallons of water per day in Manhattan and The Bronx. The knowledge that there is a water meter in the cellar is a powerful reminder to have leaky plumbing fixed and shows up a marvellous number of unsuspected leaks.*

* I venture to give two or three examples of good citizens, possessed of the very best of intentions, wasting large quantities of water unknowingly, where the waste would probably not have been discovered without meters. A hundred similar illustrations could be presented.

At the Engineers' Club in New York, a few years ago, a meter was applied ; the water bill presented appeared out of all reason, but the meter was found recording correctly. After puzzling over the matter for some time, for the plumbing inside the house appeared tight, it was found that a stream had been constantly running day and night into a house tank and through the overflow pipe from the tank into the sewer. Stopping this leak reduced the quarterly water bills 66 per cent.

A very prominent mechanical engineer, Mr. Robert Cartwright, of Rochester, being of an investigating turn of mind, had a water meter put on the service pipe in his residence in May, 1895, and started to make notes on the water required for his family. The meter quickly ran up such

Certain leaks and losses too small for detection singly must continue to exist under even the closest supervision of leaks in mains and service pipes, and with a meter on every tap. These in the aggregate cannot probably be brought below 5 to 10 gallons per inhabitant per day. In Fall River and a few other cities where meters are almost universal, the discrepancy between what is measured into the main pipes at the pumps and what is measured out from the small service pipes by the house meters, etc., is found to be anywhere from 6 to 20 gallons per inhabitant per day.

In the Deacon meter record shown in Diagram No. 4, it will be noted that after rigorous inspection and thorough repair of main and service pipes in the street and fixtures in the house, the total night use, which was doubtless substantially all waste in this small district, was not brought below a rate of 10 gallons per inhabitant per day.

Under equal qualities of workmanship in one city as compared with another, this leakage from mains and services should be in direct proportion to the diameter and length of the pipes and the number of service pipes, and since the number of miles of pipe per thousand of population is far less in New York than in any other American city, we may reasonably expect that the incurable waste, after complete metering and thorough repairs, would be found as low in New York City as anywhere; Fall River has succeeded in bringing the "consumption unaccounted for" down temporarily as low as 7 gallons per inhabitant per day, but 10 gallons is as low as this can probably be brought in New York as a whole, for any considerable term.

Probable Subdivision of Use and Waste.

It is of interest when trying to forecast the future and to decide how far we are likely to get in checking waste, to assign the most definite value that we can to each class of waste. As has already been stated, there is no means of knowing at present just how the different uses and wastes are subdivided in New York. From such information as we have from investigations made in other cities, and starting with the Central Park observations, which are described in Appendix No. 3, I venture to present the following, with the caution that these smaller subdivisions of the waste rest on opinion and not upon demonstration, and that other engineers of broader experience than I possess might adjust the proportions differently.

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a record that he thought that one of his good friends, a manufacturer of water meters, was playing a practical joke and had furnished him a meter with an extra gear in the register, which showed ten times the real use. This explanation failed, and after considerable search it was found that a by-pass valve in the house piping, which was always kept shut, had been sent from the factory without a valve-seat and had been for two years passing water almost as freely as if open, and thus a steady stream had been running through this into the sewer for two years.

TABLE NO. 2.

Probable Distribution of Use and Waste—Present and Prospective.

For the Borough of Manhattan Only.

This table is to be used with caution as to subdivision of waste.

While the totals and subtotals in the first column are very nearly right its subdivisions and also the second and third columns are merely probabilities, and rest largely upon experience in cities other than New York.

Several of these subdivisions are presented merely as matters of opinion, subject to revision after further investigation.

It is to be noted that the number of water-pipe joints per 1,000 inhabitants and the number of service-pipes per 1,000 inhabitants is not half as great in New York as in most American cities; whence the gallons per capita leaking outside the cellar walls in New York should be relatively small.

	As matters stand today in Manhattan and The Bronx.	**Ideal conditions** after thorough overhauling, with a meter on every tap, with most thorough inspection practicable, and with prompt repairs.	**Best result that can be reasonably expected** with domestic meters **10 years hence,** starting with thorough overhauling and such subsequent management as can probably be secured judging from experience of the average large American city.
	Gallons per inhabitant per day.	Gallons per inhabitant per day.	
Actual use—			
Domestic (average)	12 to 20	About the same as now.	13 to 20
Manufacturing and commercial	20 to 30		25 to 35
City buildings, etc.	2 to 4		2 to 4
Fires and street flushing and sprinkling	0.4 to 0.7		0.4 to 0.7
Total	34 to 55	34 to 55	40 to 60
Incurable waste (probabilities)—			
Leaks in mains	1 to 2	About the same as now.	A slight increase as pipes and fixtures average older.
Leaks in old and abandoned service pipes	1 to 2		
Poor plumbing, with all taps metered and close inspection	2 to 3		
Careless and willful waste	1 to 2		
Under registry of meters—Commercial	1 to 1		
" Domestic	0 to 0	1 to 1	0 to 1
Total incurable and under registry.	6 to 10	8 to 12	10 to 17
Minimum use and waste	40 to 65	42 to 67	50 to 77
Needless curable waste—			
The following subdivisions are merely assumptions and do not rest on definite knowledge. The total is fairly certain:			
In street—Leaks in mains (for a guess)	15 to 10	None.	5 to 10
" Leaks in service pipes "	15 to 10		3 to 5
In buildings—Defective plumbing "	25 to 15		15 to 20
" Careless and willful "	17 to 14		10 to 15
To prevent freezing in winter / Mean for en-			
To get cooler water in summer \ tire year.	3 to 1		1 to 3
Total needless waste	75 to 50	0 to 0	34 to 53
Total consumption	115 to 116	42 to 67	84 to 130
Complete Meters alone will probably cure only about half the waste.			Without complete metering. Or probably about 75 to 100 with a meter on every tap.

It must be borne in mind that in a large city, the houses of the working people are far more numerous than the homes of luxury, and that the experience in cities where all taps are metered is that *the per capita use of water in the cheaper houses is much less than in the more expensive houses.* The following table (No. 3) taken from a report made by Dexter Brackett, C. E., as a part of the preliminary investigations for the Massachusetts Metropolitan Supply in 1894, is of much interest in this connection.

Quantity Required for Domestic Use.

In support of the subdivisions of the preceding table, particularly of the low figure set down for the actual domestic use, exclusive of waste, namely:

Present actual domestic use.....12 to 20 gallons per inhabitant per 24 hours.

Brief consideration will show that the only reliable guide is the experience of some city of considerable size in which nearly every tap is metered, and in which the records of domestic consumption are classified separately from other uses. Any estimate based upon a small number of selected dwellings in New York is likely to err in the relative number of houses of each class and the per capita consumption probably averages from 2 to 10 times as great in middle class and high class houses as in the more humble dwellings.

The following are the best data found available for the average domestic consumption of water in large communities having a large industrial population:

In Fall River in the year 1891, 70,000 inhabitants drew, as shown by the sum of the domestic meter readings, an average of .....11.2 gallons per inhabitant per day.

In Fall River in 1899—the Water Registrar has very kindly made up for me a separate classification of the domestic meter accounts, which shows the total consumption by a population estimated at 99,000 to have averaged—in the year 1899 .....14.6 gallons per inhabitant per day.

In Lawrence, Mass., another city largely devoted to manufacturing, a compilation recently made for me in the office of the Water Board, shows that the average metered consumption by that portion of the population having the metered services, estimated at 43,825, averaged.....16.3 gallons per inhabitant per day.

In Woonsocket, R. I., another city where substantially every tap is metered, the domestic consumption of 27,500 population, from the aggregate of the metered readings, as kindly compiled for me by the Superintendent, Byron W. Cooke, C. E., averaged for the year 1899.....14.1 gallons per inhabitant per day.

In Woonsocket domestic consumption averaged in 1898.....13.4 gallons per inhabitant per day.

In Worcester, Mass., for the year ending November 30, 1899, with substantially every service pipe metered, the total drawn through the domestic meters during the year by a population of about 112,000, averaged .....14 gallons per inhabitant per day.

In the cities of Providence and Pawtucket, also large manufacturing cities, although nearly every domestic tap is metered, the records of the Water Registrar's office are not kept in such a way that the average domestic consumption per capita can be learned without great labor.

Each of the above figures for Fall River, Worcester, Lawrence and Woonsocket it must be remembered includes such domestic waste as the average man permits to exist when he has a water meter in the cellar to quicken his memory and his interest in the repair of leaks.

In suburban communities, where meters have perhaps been more common than in the large cities, and where there are lawns to be watered and carriages to be washed, the average meter consumption per capita runs higher, but for the average of the great mass of the population of Manhattan and Brooklyn, I believe the figures adopted in Table No. 2 are not far out of the way.

A careful analysis of meter readings on many hundred houses made by Mr. Dexter Brackett during the investigations preliminary to the work of the Massachusetts Metropolitan Water Board showed that the actual use of water per inhabitant was very much less in the more humble houses, or the houses with a single faucet, than in the houses of the well to do or wealthy. For example, the average of 619 families in the cheaper grade of houses in the City of Newton, Mass., having but one faucet each, showed a daily consumption of only about .....7 gallons per inhabitant per day.

TABLE NO. 3.

Consumption per Capita for Domestic Use in Boston, Brookline, Newton, Fall River, Worcester, and London, England, as Determined by Meter Measurement. Compiled by Dexter Brackett, C. E., November 1, 1894.

	Number of Houses.	Number of Families.	Number of Persons.	Metered Consumption, Gallons per		Remarks.
				Family.	Capita.	
Boston	31	402	1,461	221.	59.	Highest cost apartment houses in the city.
"	46	628	2,524	185.	46.	First-class apartment houses.
"	223	2,204	8,432	123.	32.	Moderate-class apartment houses.
"	39	413	1,844	80.	16.6	Poorest-class apartment houses.
"	339	3,647	14,261	139.	35.6	Average of all apartment houses supplied by meter.
"	40	.....	1,699	.....	46.1	Boarding houses.
Brookline	....	828	4,140	221.5	44.3	Average of all dwellings supplied by meter.
Newton	490	490	2,450	132.5	26.5	All houses supplied with modern plumbing.
"	....	619	3,005	.....	**6.6**	These families have but one faucet each.
"	....	278	1,390	34.5	**6.9**	These families have but one faucet each.
Fall River	28	34	170	127.5	25.5	The most expensive houses in the city.
"	64	148	740	42.	**8.4**	Average class of houses, generally having bath and water-closet.
Worcester	....	20,514	90,942	.....	**16.8**	Whole domestic consumption.
"	....	81	327	80.2	19.9	Woodland street, best class of houses.
"	....	37	187	118.1	23.4	Cedar street, best class of houses.
"	....	93	447	95.	19.8	Elm street, houses of moderate cost.
"	....	245	1,104	55.1	**13.2**	Southbridge street, cheaper houses.
"	....	229	809	55.	**15.6**	Austin street, cheaper houses.
London (England)	1,169	.....	1,183	.....	25.5	Houses renting from $250 to $600, each having bath and two water-closets.
"	727	.....	5,089	.....	18.6	Middle class, average rental $200.

(NOTE BY J. R. F.—In considering the average domestic use of New York or Brooklyn with their millions of inhabitants *on the per capita basis*, we must bear in mind the overwhelming preponderance in number of the cheaper houses in each of which the number of fixtures through which water may be drawn or wasted is comparatively small; otherwise the preponderance in number of tests upon the better grade of houses may mislead.)

Manufacturing and Commercial Use.

In Water Commissioner Dalton's Report for 1898, our best authority for the estimate in Table No. 2, of 20 to 30 gallons per day for the present actual use for commercial and manufacturing, is the statement, page 9, that the present 35,442 meters in Manhattan and The Bronx "*cover every place where water is used to any considerable extent for other than domestic purposes.*"

An analysis of figures given me in the office of the Water Register, presented in Table No. 4, on pages 52 and 53, show that for several years past the consumption of metered water in proportion to the total population of Manhattan and The Bronx has averaged..... 24 gallons per inhabitant per day.

This metered quantity includes also a considerable quantity used by hotels, clubs, and a few private houses and some tenements having stores beneath.

This 24 gallons per inhabitant covers the use for New York's shipping, railroads, manufacturing, tall office buildings, electric stations, and a large part of the transient population. I had expected the use for manufacturing and commercial purposes would prove larger partly because of the large metered use for this purpose in Boston, and in view of the fact that New York City is one of the chief manufacturing centres in this country, although the individual factories are small, that it is surrounded by salt water, with no fresh water streams like most of our manufacturing centres, so that its thousands of steam engines are mostly of a type requiring large quantities of water, and in view of the uncommonly large requirements for water in New York for shipping and railroad purposes. The explanation may be that, dividing the actual quantity by the enormous population of about two millions, makes the *per capita* figure unexpectedly small.

I have no good reason for doubting the accuracy of the statement above quoted that substantially all water for commercial purposes is now metered, and have no occasion for doubting and no means for checking up the totals for metered water in Table No. 4, given me in the Water Register's office, and think the domestic use metered should offset under registry, yet this figure of 24 gallons per capita for the metered use for all commercial and manufacturing purposes does look small.

The only apparent margin of uncertainty is that perhaps under registry of meters and underestimate for special uses might bring the total up to 30, while deductions for hotels, clubs and the few domestic meters might take it down to 20 gallons per capita per day.

TABLE

Average Daily Use of Metered Water Each Year

Year	Actual cash receipts from sale of metered water, exclusive of Riverdale and steamboat meters, as stated or by Mr. Arthur Miller, chief clerk of Bureau of Water Register.	Receipts from dock steamboat meters.	Charges for metered water remaining unpaid and turned over to Department of Arrears for collection, as found in recapitulations on books of Bureau of Arrears by Mr. Maher.	Total value of water metered, exclusive of Riverdale supply from Yonkers, as found by adding preceding 3 columns.	Corresponding amount of water metered at $1 per 1,000 cubic feet. — Cubic feet per year. — (All metered water is sold at same price.)	Corresponding average number of million gallons per day of metered water drawn. — ($1 per year pays for 20.5 gallons per day.)	Total number of meters in use December 31, per printed reports of Deoartment Public Works.
1889	.....	.....	$114,384*	.....	.....	.....	.....
1890	$915,276 917,288*	$78,224*	160,465	$1,153,905	1,153,905,000	23.7 28.5*	22,672
1891	1,219,055 1,307,732*	85,923*	289,546	1,594,524	1,594,524,000	32.6 30.5*	24,264*
1892	1,232,479	81,355*	151,857	1,465,691	1,465,691,000	30.0	27,041
1893	1,832,889	99,474*	182,019	2,114,382	2,114,382,000	43.3	30,286*
1894	1,659,110 1,662,917*	99,660	263,542	2,022,312	2,022,312,000	41.5 36.6*	30,325*
1895	1,766,690	Abt. 100,000	264,414	Abt. 2,071,104	Abt 2,070,000,000	42.4	.....
1896	1,891,480	105,331*	296,759	2,293,570	2,293,570,000	47.0	33,571
1897	2,012,118	85,525	244,523	2,342,460	2,342,466,000	48.1	34,590
1898	2,070,514	109,438	207,790	2,387,762	2,387,762,000	49.0	35,442
1899	.....	.....	.....	.....	.....	.....	.....

*From printed reports of Dept. Public Works for that year.
†From printed report Dept. Water Supply 1898.
Note that the arrears stated yearly in Water Register's report in Appendix to printed report of Dept. The printed Dept. Public Works reports for 1890–1–4 contains table showing distribution of metered water Apparently these estimates do not include the meter rates returned to Bureau of Arrears for collection.
Meters in 1897 found by 35,142—352 = 34,590, deducting meters placed in 1898 from total at end of 1898.

No 4.

in Territory Supplied from Croton and Bronx.

Total revenue of Water Department including the amounts uncollected by Register Department and returned to Bureau of Arrears for collection per D. P. W. reports.	Per cent. of total water metered.	Per cent. of total revenue derived from metered water.	Total consumption per day in million gallons.	Population supplied, from estimates of Health Department.	Gallons supplied per day per inhabitant.	**Gallons of metered water per inhabitant per day.**	Gallons of unmetered water per inhabitant.	Total revenue of Water Department per inhabitant.	Approximate revenue from domestic service per inhabitant, found by dividing total revenue, less that from metered water by population.
$2,782,051†	....	....	...	.....	....	....	....	....	.....
2,980,005†	19.9	38.8	119	1,711 000	69.6	**13.8**	55.8	$1.74	$1.07
3,497,848†	21.3	45.6	153	1,755,000	87.3	**18.6**	68.7	1.99	1.08
3,275,411†	18.4	44.8	163	1,802,000	90.5	**16.6**	73.9	1.82	1.00
3,928,520†	24.7	53.8	175	1,818,000	96.4	**23.8**	72.6	2.16	1.00
3,799,679†	23.6	53.2	176	1,835,000	96.0	**22.6**	73.4	2.07	.97
3,938 303†	24.1	52.7	180	1,851,000	97.3	**22.9**	74.4	2.12	1.00
4,223,181†	23.4	54.3	201	1,934,000	103.8	**24.2**	79.4	2.18	1.00
4,334,483†	22.7	54.1	212	1,990,000	106.5	**24.1**	82.4	2.18	1.00
4,459,404†	21.7	53.7	226	2,040,000	110.0	**23.9**	86.1	2.18	1.00
.....	....	....	246	2,117,000	116.0	....	....	....	.....

Public Works are for year previous.
among different classes of establishments, also different sizes of meters in use.

Special Needs for Water in New York.

Considering further the reasonableness of the allowance made in Table No. 2 of 20 to 30 gallons per capita for use for commercial and manufacturing purposes, and also considering the actual records of Table No. 4 of metered consumption in the light of what has been found elsewhere:

In Boston, for example, according to the very complete analysis made by Mr. Dexter Brackett in 1894 precedent to the establishment of the Metropolitan Water Board (see Rept. Mass. State Bd. of Health, Met. Water Supply, 1895, p. 167). The total use of water for commercial and manufacturing purposes in Boston in 1892 was about 30 gallons per capita per day. I have some question if count of population used in figuring this per capita is strictly comparable with that in New York, because of the probable larger proportionate number of population living in the suburbs of Boston and thus not counting in the enumeration and whose omission raises the per capita figure.

Briefly considering various causes affecting New York, we find:

1st. Of non-resident users. Large numbers of persons in Manhattan by day use water very freely in their offices or in their workshops, or require the use of water for the preparation of their mid-day meal, but are not counted in the population because they spend the night elsewhere. Trying a little arithmetic on this, we find that if there are 300,000 such from Jersey and beyond the Harlem and across the East river, and if each average using half of his proper daily supply of 13 gallons in Manhattan, it would add only 1.0 gallon per inhabitant per day to the per capita use of the resident 2,000,000, and this extra gallon would be found almost entirely in the portion now metered.

New York, as the commercial centre of America, has a large floating population which constantly uses water, but largely escapes the census enumerator. It has been estimated, with what accuracy I do not know, that this class numbers upward of 200,000, an addition of 10 per cent. to the enumerated inhabitants. Here again a 10 per cent. addition to the average quantity really used for domestic purposes would be only an addition of 1.3 gallons per capita on the enumerated population, and much of this extra use would be found in hotels which are metered.

2d. Use for steam and manufacturing. New York is said to be one of the largest manufacturing centres in America in number of hands employed, but the power required is probably relatively small per operative. Unlike most other manufacturing centres, all the water that surrounds its territory is salt. Its thousands of steam engines, the most of them of a small type, using abnormally large quantities of water per horse-power, have to take water for steam-boiler supply from the public water-mains. The vast majority of these steam plants have no opportunity for water for condensation.

It appears probable that in in New York more than in most other cities, there will abound a variety of smaller industries which use water freely, such as printing offices, lithographers, photographers, chemists and a thousand others.

But the Annual Report of the Department of Water Supply for 1898, page 9, states that *the present* 35,442 *meters in Manhattan and The Bronx "cover every place where water is used in any considerable extent for other than domestic purposes,"* and therefore in following Table No. 4, which finds a per capita use for all metered water of 24 gallons per capita per day, we include the actual quantity taken for these special uses.

Preceding Table, No. 4, shows that this use of water for manufacturing and commercial purposes averaged 24 gallons per inhabitant per day, and presumably the number of hotels and apartment houses and clubs included in this would more than offset any under registry of these meters, leaving 24 gallons as the fair allowance for all manufacturing and commercial use.

Some very interesting data on this increased use by tall buildings were collected with the assistance of the Water Registrar.

It was found, for example, that the consumption by the American Tract Society Building averaged about 63,000 gallons per day, whereas the old buildings formerly on the same site averaged only 6,600 gallons per day, or about one-ninth part as much.

The New York Life Building uses two and one-half times as much as the buildings formerly on the same site.

The Empire Building and the World Building, nearly twenty times as much.

These are, of course, extreme examples; but all such increase of use is understood to be covered by the meter records of Table No. 4.

3d. The use of water by the shipping, and for the railroads, is probably out of all proportion to the per capita use for such purposes in other large cities. This again is part of the 24 gallons of metered water per inhabitant, except that towboats are understood to water or "Croton" at any convenient hydrant on the water-front under an annual payment of $90.

4th. Tall buildings require more water than low buildings; increased elevator service means more power and more water for steam. This is also covered under the 24 gallons of metered water per inhabitant per day.

Allowance for Consumption of Water in City Buildings, etc.

The minimum figure of 2 gallons per capita entered for this use in Table No. 2 presupposes economy and is the quantity that would be drawn if there were a water meter on each public building, each school-house, fire-engine house and public urinal, and if comparative records kept and looked after, and any extravagance or undue waste promptly checked; that is, the minimum figure represents use proper while the larger figure of 4 gallons per capita represents such free use as may be expected under good, ordinary management without any special effort toward economy.

That these figures are reasonable is confirmed by the records of the amounts of water actually used for these purposes in Boston in 1892, as compiled by Dexter Brackett, C. E. Mr. Brackett's analysis may be found in Report Massachusetts State Board of Health on Metropolitan Water Supply, 1896, pages 167 and 169. A part of these uses in Boston were not metered but taken by estimation. These figures of Table No. 2 are also confirmed fairly well by the investigations made into the use by public buildings in Fall River, in the effort to account for all water used, and restrict waste; also by similar records in Worcester, Mass. The great concentration of population in New York should tend to keep this use small as reckoned on the per capita basis.

Street Sprinkling.—The total use for fires and for street sprinkling is much smaller than most persons realize, and the quantity used in the most liberal flushing of streets heretofore carried on in New York is also very small in proportion to the other uses. So small, indeed, that they cut no appreciable figure in the question of magnitude of supply needed for the city as a whole, and if the entire volume needed for these purposes were to

be salt water taken from the harbor this would not defer the time or lessen the magnitude of the new supply that must be sought.

Some vigorous experiments in street flushing in New York at night were undertaken by the late Col. George E. Waring, C. E., Superintendent Street Cleaning, for about six weeks, beginning about August 1, 1896, which are of interest as illustrating the largest use of water for this purpose yet made in New York. George A. Taber, C. E., my assistant on much of this work, was a member of Col. Waring's staff and Superintendent of the Seventh District, and has made up as good an estimate as now possible from his own experience and by conference with nearly all of the other district superintendents, as to the size of nozzle, number of streams in use, and number of hours' use per night, from which it appears that the total use in all ten districts average about 1,000,000 gallons per day during the month or six weeks that the experiment lasted. The flushing gangs began about midnight, worked until 5 A. M., and in each district a 1-inch nozzle, under full hydrant pressure, was kept going about 2½ hours per night, using at each nozzle about 25,000 gallons per night. Much more of this work was done in the Second District than eleswhere, and in the Second, Fourth and Fifth Districts together, more than half the entire quantity estimated for the entire city was used, the Second District being in the quarter thickly populated by the poorer Hebrews. I am informed that such free flushing of pavements has never been undertaken before or since, and it will be noted that this, in fair, warm weather, only required about 0.5 gallons per capita per 24 hours.

The total use of water for street sprinkling in the year 1898 may be inferred from the revenue as given on page 35 of the Report of the Department of Water Supply for 1898. The total revenue from this source in Manhattan and The Bronx is given as $28,000 for the year. If this water was sold at the regular price for metered water this would represent 28,000,000 cubic feet, or 210,000,000 gallons for the year, an average for the year of about 600,000 gallons per day, which, divided by 2,150,000 population gives 0.28 gallons per capita per day.

In Boston, Mr. Brackett's estimate was	1.0 gallon per capita.
In Fall River, in 1899, this averaged for the year	1.0 "
In Worcester, Mass	1.9 "

The great concentration of population, and the much less length of street per 1,000 of population, tends to make the per capita use for this purpose relatively small. The figures 0.4 to 0.7 gallons per capita per day, adopted for this use, appear proper, in view of the record of 0.28 gallon per capita for sprinkling and 0.07 for fires during the past year.

Fires.—The Fire Department records show that last year less than 50 million gallons of Croton water was used at fires during the entire year. This corresponds to a per capita use of 0.07 gallons daily. In the two preceding years it was only two-thirds of this amount.

In Boston Mr. Brackett estimated the use for fires according to the best records available at 0.10 gallon per capita per year.

Total Actual Use of Water.

By adding together the several classes of use already discussed and set down in Table No. 2, we obtain for the probable actual use in Manhattan and The Bronx, as matters stand to-day, figures varying from 34 to 55 gallons per capita per day, according as the smallest possible value or the largest reasonable value is sought. No reasonable ground appears for think-

ing it less than 34 or greater than 55, and the best evidence is that it is between 35 and 40 gallons per inhabitant per day.

No particular attention appears to have been paid in the Water Register's Department to classifying records so as to show how the water paid for has been used, and no particular attention given by the Engineers to taking measurements for finding out how much of the water delivered was paid for, or just where it went, and the records of the department are in this regard much inferior to those of Fall River, Worcester and some other cities; even the very interesting and useful analysis and classification of the meter receipts presented for some years by Mr. Birdsal, one of which is printed in page 87 of the Department of Public Works Report for the last quarter of 1894, has been discontinued on the transfer of this account to another bureau. We therefore have to rely largely on data from other cities and upon the measurements made at the Central Park reservoirs, already referred to.

The Central Park experiments, as already stated, appear to demonstrate conclusively that the quantity actually used for all purposes, apart from all waste, does not exceed 40 gallons per capita per day.

The reasonableness of this figure is confirmed by experience in cities where an earnest effort has been made to reduce waste by the application of a meter to nearly every tap as follows:

Results of Meters in Saving Waste.

In Fall River, Mass., with a population estimated at 102,281, mainly engaged in manufacturing, but with the water supply for steam engines, washing and privies in the large factories, mainly drawn from the river and ponds, the total consumption measured for the year 1899, including domestic service, factory services and public uses, and with a pressure in the mains averaging 80 pounds, or double that in Manhattan, and in a large part of the city 120 pounds per square inch, averaged only (including all use and waste) ..... 34 gallons per inhabitant per day.

In Woonsocket, R. I., with population supplied estimated at 28,000, with substantially all services metered from the beginning, population mainly engaged in manufacturing, but with factory supplies taken in part from the river, the total consumption measured at the pump, with an average pressure of from 80 pounds per square inch, or double the pressure existing in Manhattan, for the year 1899 averaged (including all use and waste) ..... 29 gallons per inhabitant per day.

In Syracuse, N. Y., with population in 1898 of 124,856, the daily consumption measured at the intake from Skaneateles Lake, and thus including whatever leakage there may be in 20 miles of 36-inch cast-iron pipe between the lake and the city, and including also whatever leakage there may be from distribution reservoir pipe system, it appears by the official report for 1898 that the total use and waste, with only 44 per cent. of the service pipes metered, averaged (including all use and waste)..... 64 gallons per inhabitant per day.

The case of Syracuse is of particular interest in connection with any proposition to ultimately increase the pressure in New York, because of the pressure in Syracuse having been increased about 50 pounds, or from 45 to 95 without any radical change in house plumbing or general relaying of the old street mains.

In Providence, R. I., with a population estimated at 170,000 in 1898, with 80 per cent. of all the service pipes metered, and with a very large use for manufacturing and for steam engines, and with an uncommonly large number of hydraulic elevators connected directly to the city water pipes for motive power (a use very rare in New York), the total consumption for all purposes, and with a large proportion of high grade residences, averaging in 1898 (including all use and waste). ..... 54 gallons per inhabitant per day.

In Worcester, Mass., with a population estimated at 110,000 in 1899, with 94 per cent. of all taps metered, the total consumption shown by a Venturi meter on the supply pipe and use, averaged in 1899 (including all use and waste)...67.6 gallons per inhabitant per day.

Of this the service meters showed—

Use for manufacturing	17 gallons.
Domestic use.....	14 "
Public buildings, etc.....	2 "
Street sprinkling (free water)	2 "
Total use accounted for	35 gallons.
Pipe leakage, puddling, flushing and unaccounted for.....	32 "

(It is proposed to investigate and reduce this last item during the coming season.)

In Newton, Mass., with a scattered suburban population estimated at 30,000, many houses having large lawns and gardens and private stables, and with many houses of the class commonly found using water very freely, but with little manufacturing, with 80 per cent. of all the service pipes metered, the consumption in 1898 averaged (including all use and waste)..... 59 gallons per inhabitant per day.

This is probably fairly typical of what may be later expected in the best suburban districts of Brooklyn, The Bronx and Richmond.

Incurable Waste.

For this item of Table No. 2 we have to rely largely upon judgment, based in part on such information as can be gained from experience elsewhere.

The incurable waste arises from three principal causes:

First—The hundreds of thousands of microscopic leaks at the main pipe joints, each of which individually is too small to attract notice or to warrant the expense of a remedy, but which in the aggregate amount to a very noteworthy quantity. Manhattan and The Bronx have about 833 miles of main pipe, all of cast-iron, in pieces none more than 12 feet long, and with many added joints at special fittings; there are thus about half a million of these leaded joints in Manhattan and their aggregate length is about 250 miles.

Each joint is made by inserting the "spigot end" of one pipe into the "bell end" of the next and pouring melted lead into the narrow space between them, which lead when cooled is com-

pressed or "staked" by a heavy hammer and a calking tool so as to tightly fill the space and make a tight joint. Any one who has watched a pipe-calker work on the under side of a large pipe in a deep trench in a narrow, poorly-lighted, strained, uncomfortable position will appreciate that perfection is difficult to obtain even by skilled and faithful workers.

In the extremely large amount of heavy pipe recently laid in on the Massachusetts Metropolitan Works, all sections were tested by water pressure with ends closed before refilling the earth into the trench, and the amount of water that had to be pumped in to maintain a constant pressure was surprisingly large in view of the great care in inspecting the pipe and the precautions for securing tight joints. I have had frequent occasion to discover the extent of this underground leakage in tests of fire-pipe systems, from which no water was drawn except what disappeared through invisible leaks.

While many of the most microscopic leaks found in testing a long line of newly laid pipe will doubtless become clogged by silt, algae or rust, others are sure to open under the strains due to settlement of earth. Changes of temperature between summer and winter and by the opening up of blow-holes or spots, spongy metal hidden at first by scale or tar coating, or by incipient fractures or cracks which open a little but do not "let go." The larger leaks may saturate the ground so as to show on the surface, but there is no way of finding the thousands of smaller leaks which soak down into the earth.

Second—The service pipes between the cast-iron main and the main stop-cock inside the cellar are often a source of leakage. In New York City two-thirds of the pipes themselves are said to be of lead, which tends toward average tightness, but the "corporation cock" by which the service pipe is attached to the cast-iron main used in New York is of a type used almost nowhere else, which it would appear might be subject to dislocation or leakage. It is not screwed into the main, but consists merely of a hollow tapered brass plug, driven into a smooth hole drilled into the main and which may become loosened by a strain on the pipe.

Third—Abandoned service pipes are often a particularly fruitful source of leakage. I am told that there are perhaps 20,000 of these in the Borough of Manhattan, and that the great majority have not been cut off and plugged at the main, and that many of the abandoned lead pipes have been abandoned after merely doubling the end of the pipe near the cellar wall back on itself and hammering it together at the bend then leaving it and burying the end in the earth.

Old wrought-iron service pipes are particularly susceptible to rust and when uncovered are sometimes found leaking at the joints. These leaks are so small that individually there is no hope of discovering them or of stopping them without relaying.

Just how much of the incurable leakage comes from the joints in the cast-iron mains and how much from joints at the corporation cock and leaks along the service pipe we have no means of knowing, but from the measurements and records of certain metered cities we have good data for proving that the total of this need not exceed 5 or 10 gallons per capita per day under ordinary conditions.

Data on Incurable Leakage Outside the Cellar Wall.

Fall River, R. I., with substantially every tap metered, pressure averaging 80 to 120 pounds per square inch in the principal districts, with 85 miles of main pipe and about 100,000 population, thus presenting a concentration of population of 1,170 inhabitants per mile of pipe, and with about 1.1 lineal feet of leaded pipe joint per inhabitant. With the best that the very efficient Superintendent, Mr. Kieran, has been able to accomplish in reducing waste, the water pumped into the pipes exceeds the volume of water measured out from the pipes through the meters and other known deliveries by 8.5 gallons per inhabitant per day.

During the tests of night consumption kindly made for me by Mr. Kieran in April, 1900, the smallest night flow found averaging at the rate of about.....	13 gallons per inhabitant per day.
Of this certain drinking-fountains and watering-troughs and public urinals were found to be taking about	3 " "
Leaving for smallest rate of night use, plus leakage, about.....	10 gallons per inhabitant per day.

—the useful part of which was, of course, caught by the meters; also the leakage of house plumbing. Therefore, probably the main-pipe and service-pipe leakage combined could not have exceeded 8 gallons per inhabitant per day.

Woonsocket, R. I., has substantially every tap metered, pressure averages about 80 pounds per square inch; has 45 miles of main-pipe and about 28,000 population supplied, thus presenting a concentration of population of 620 inhabitants per mile of pipe; works 16 years old; works are managed with remarkable efficiency in the prevention of waste by Byron W. Cook, C. E., Superntendent. The total discrepancy between the water pumped into the stand-pipe (with 2 per ent. allowed for slip) and the water metered out of the distribution pipes, after allowing for a few nmetered uses, but with no allowance for under registry:

Average in 1898, disappearance from mains.....	8.5 gallons per inhabitant.
" 1899, " "	7.8 "

During the test of night consumption very kindly made at my request in May, 1900, the average night draft at the minimum hour was at the rate 9 gallons per inhabitant per 24 hours. This night was very accurately measured by the drawing down of the stand-pipe levels, the pump being stopped at 6 P. M. Some small part of this night consumption was no doubt used, and leakage from plumbing, that would be caught by the meters, so that the pipe leakage could have hardly exceeded 6 gallons per inhabitant per day.

Boston, Mass. After the very rigorous inspection for detecting waste made in certain districts in 1884, by the aid of the Deacon meter, and after thorough repair of all leaks found, there still remained a considerable amount of incurable leakage. For example, in Fig. 7, transcribed from the Deacon meter records from two of the Charlestown districts, it will be noted that the night use was not brought below a rate 10 gallons per inhabitant per 24 hours.

In Providence the minimum night consumption, at the rate of 20 gallons per capita, shown by the line on Fig. 5, included the actual night use, and is also understood to include the possible leakage from two large distributing reservoirs with earth banks and beds.

In Milton, Mass., in a small scattered suburban community, with only about 300 inhabitants per mile of pipe, with all the water metered into the main pipe, and with a meter on every service-pipe leading out, it is reported that a few years after the works were built a comparison of the water measured in with the water measured out showed about 27 gallons per capita unaccounted for. After a careful inspection and remedy of leaks in main-pipes and service-pipes this quantity unaccounted for was reduced to about 3 gallons per capita per day, a remarkably small amount considering the small population per mile.

Since writing the above Mr. Ellerton P. Whitney, President, Milton Water Company, informs me that the superintendent, by a frequent midnight inspection of the supply meter, and cutting out of one section or another of the pipe system to locate leaks, has so cured this leakage that it is frequently found so small that the large meters cannot register it, and apparently the entire waste from the pipe system is now less than five per cent. of the consumption or probably less than two gallons per capita per day.

A more appropriate basis than the per capita basis for comparison of this kind of leakage is to make the comparison on the basis of gallons per

Additional Data on Tightness of Main Pipes.

Test of Providence Special High-Pressure Fire Pipe System.

Reliable data on main pipe leakage free from complication with service pipes or other connections is difficult to obtain, particularly for pipe long enough in use to have the microscopic cracks rusted and silted up. The very best piece of data that I have been able to obtain showing what is possible in tightness, has come to hand June 10, 1900, after the last proof-sheet was returned, and is as follows:

The City of Providence has a special fire protection pipe, now about three years old, 5.57 miles long, mainly 16 inches diameter, under an average pressure of 114 pounds per square inch in the business section. This pipe has no connections of any description except fire hydrants and a waste pipe. A test of the tightness of this pipe was very kindly made for me by Mr. Otis Clapp, City Engineer of Providence, June 10, 1900, by tapping a small water meter in on a by-pass around the main gate. There is good reason to believe the main gate substantially tight, while there is possibly some leak from the 4-inch gate between high and low pressure; probably more than enough to offset any leak at main gate.

The test shows a leakage for the entire 5.57 miles at rate of.....	2,487 gallons per 24 hours,
which is.....	446 gallons per mile per 24 hours,
equivalent to.....	0.22 gallons per foot of leaded joint per 24 hours;
and with equally tight street mains in New York, the loss would be from street mains only.....	0.13 gallons per inhabitant per 24 hours.

This, with the Milton tests (see page 287), indicates that my suggested ideal of 5 to 10 gallons per capita for main-pipe leakage in New York permits more leakage than necessary.

(See also page 287.)

day lost per mile of main pipe, and a more accurate basis still would be the number of lineal feet of leaded joint plus some factor dependent upon the number of service pipes. Because all our discussion of Table No. 2 is on the per capita basis, we will retain that, but will take note of the different density of population in the several communities compared.

	Average Working Pressure. Lbs. per Sq. in.	Approx. Miles of Main Pipe.	Approx. Population Supplied.	Approx. Population per Mile of Main Pipe.	Approx. Length of Leaded Joint, per Capita.	Approx. No. of Service Pipes.	Population per Service Pipe.
Manhattan and Bronx.	35	833	2,080,000	2,500	0.6	124,000	17
Brooklyn (Ridgewood)	45	540	1,114,000	2,060	....	110,000	10
Fall River.....	80	86	99,000	1,150	1.1	6,800	14
Worcester	80	165	110,000	660			

Additional Data on Tightness of Main Pipes.

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Additional data received June 11, 1900. Attach to page 60 (a), Report N. Y. W. S.

Tightness of Certain New Lines of Large Water Pipe.

(From data furnished by Dexter Brackett, C. E.)

In the following table are given results of tests upon pipe lines recently laid by the Metropolitan Water Board under contract, according to carefully drawn specifications, under the supervision of F. P. Stearns, Chief Engineer, and Dexter Brackett, Engineer of Distribution Department.

These tests were made soon after the pipes were laid and before they were placed in service. In a number of cases leaks were discovered and repaired after the tests, so that the leakage at the present time is probably smaller than the figures given below.

These pipes were tested under pressures of from 50 to 150 pounds per square inch, which were considerably in excess of the pressures maintained in regular service.

Inside diameter of pipe in inches.	Approximate length of leaded joint per linear foot of pipe line feet (as computed by J. R. F.)	Sum of lengths tested.		Average leakage found per linear foot of pipe in gallons per 24 hours.	Average leakage per linear foot of lead-joint in gallons per 24 hours.	Product of leak per foot of lead-joint by total length of such joints.
		Feet.	Miles.			
48	1.17	116,563	22.0	3.7	3.16	426,000
42	1.03	11,744	2.2	2.8	2.43	29,200
36	0.91	25,663	4.9	2.9	3.19	74,500
30	0.74	7,287	1.4	0.6	0.81	4,360
24	0.61	20,553	3.9	2.1	3.44	43,100
20	0.51	40,231	7.6	3.6	7.00	143,500
16	0.41	49,903	9.4	1.9	4.68	95,000
		Total, 271,944				815,660

Average leakage per lineal foot of lead-joint per 24 hours, 3.0 gallons.

It will be noted that the leakage found in the above tests was not always in proportion to the diameter of the pipe.

At the above average rate, the main-pipe leakage in Manhattan would be only about 2 gallons per capita per 24 hours.

day lost per mile of main pipe, and a more accurate basis still would be the number of lineal feet of leaded joint plus some factor dependent upon the number of service pipes. Because all our discussion of Table No. 2 is on the per capita basis, we will retain that, but will take note of the different density of population in the several communities compared.

	Average Working Pressure. Lbs. per Sq. in.	Approx. Miles of Main Pipe.	Approx. Population Supplied.	Approx. Population per Mile of Main Pipe.	Approx. Length of Leaded Joint, per Capita.	Approx. No. of Service Pipes.	Population per Service Pipe.
Manhattan and Bronx.	35	833	2,080,000	2,500	0.6	124,000	17
Brooklyn (Ridgewood)	45	540	1,114,000	2,060	....	110,000	10
Fall River.....	80	86	99,000	1,150	1.1	6,800	14
Worcester	80	165	110,000	690	....	13,000	
Woonsocket.....	80	50	28,000	560	....	2,100	13
Providence	70	315	170,000	540	2.7	20,500	8

From the figures of the above table it is plain since the pipe leakage may fairly be assumed proportional to the length of pipe to the number of service pipes and to the pressure that, judged by each and all of these standards, the pipe leakage reckoned on the per capita basis should, with equal skill and thoroughness of workmanship and material, be only from one-fourth to one-half as great as in the cities for which we have data for comparison.

And since it is proved feasible to bring the total water unaccounted for, which includes the pipe leakage down to about 8 gallons per capita per day in the places named, it should with equal workmanship, quality of material and skill in management be possible to reduce this incurable leakage in Manhattan and The Bronx to a total of 3 to 4 gallons per day.

As against this we must take notice of the more crowded condition of the streets in the business part of the city, their more frequent disturbance by excavation for all sorts of enterprises, and, perhaps, most important of all, when considering the long future, the danger of electrolysis with the ever increasing use of electric currents. There is also the possibility that because of the congestion of traffic repairs will be less promptly made, but taking all these matters into account the totals adopted in Table No. 2 appear to be as large as there is any real excuse for.

This is lost to the city and is therefore treated as waste,

Under Registry of Meters.

The extent of under registry of meters is largely a question of local custom ; some Superintendents prefer that new meters should average under-registering 2 per cent., so that all margin of doubt shall be in favor of the customer. With meters owned by the consumer, and not systematically taken in for test and repair except at the consumer's request, or in case of a violent change

of record (as is now understood to be the practice in New York), the average of under-registry will be different from that in cities where meters are owned by the city and called in for test after passing some definite quantity, say 100,000 cubic feet for a ⅝ meter, or after 3 to 5 years of use. A meter kept out too long may become sluggish in catching the minute deliveries caused by leaks in plumbing or in the last pint delivered as the ball-cock slowly closes.

From an examination of a few typical records in the meter testing departments of several cities, and from conference with the experienced men in charge of the testing, it appears entirely practicable to maintain the under-registry of meters of the best modern types after they become 5 to 10 years old, inside of 2 to 5 per cent., and that taking the average of old and new and for all uses, 2 per cent. should be a fair average if the meters are systematically looked after.

With meters as they exist to-day in Manhattan, considering the various makes as set forth in a table in the Report of Department of Water Supply for 1894, page 88, I would not be surprised to learn that the under-registry as a whole was 10 per cent. In this I am guided by the experience of Dexter Brackett, C. E., with similar types of meters in Boston some years ago.

Subdivision of the Needless Waste.

We are, as I believe, certain that the needless waste is correctly given in Table No. 2, at from 50 to 75 gallons per capita per day, in Manhattan and The Bronx.

This is demonstrated by the Central Park experiments and confirmed by the data given above from the experience of other cities, but when it comes to subdividing this waste and saying how much comes from the plumbing and how much comes from old pipes outside the cellar wall, we have very little clear-cut evidence to guide us, and none whatever from New York itself. The subdivision of the 50 to 75 gallons into

Leaks in mains	10 to 15 gal. per cap. daily.
" " Service pipes	10 to 15 " " " "
" " Defective plumbing	15 to 25 " " " "
Careless and willful	14 to 17 " " " "

—was made in view of what has been said about service pipes of lead and iron—services abandoned without plugging off—old mains cast on the side—and some study of records of waste reduction in other cities, but may as well be presented as a "guess tempered with judgment," as to offer reasons in detail.

The figures for summer and winter waste given in Table No. 2 are deduced from noting that for several years past the line of mean consumption for each month given in Figure 1 averages

About 8 million gallons above the mean for 2 months each winter.
5 " " " " " " 2 " " summer.

These extra quantities, if spread over the 12 months, would average about 4.3 million gallons per day or 2.15 gallons per capita per day that may be fairly attributed to the waste from leaving faucets open on cold

nights in winter to prevent freezing, or from letting the water waste from the faucet in hot weather in the effort to get the water cool.

The effect of this last is obscured on the diagram by the closing during the hot months of very many of the houses of the class in which most water is used, and on the other hand, by the legitimate increase of use that comes with hot weather. The chief purpose of entering this waste as a separate line in Table No. 2 is to show that it does not form a large part of the total waste for the year.

Service Pipes.

Bearing upon the question of leaky service pipes, and of the material as affecting the durability, in response to my request for an approximate estimate, Mr. Birdsall says that of all service pipes (large and small) in—

> Manhattan, about one-third are wrought iron. Remainder of lead.
> Bronx, about one-third are wrought iron. Remainder of lead.
> Brooklyn, about one-tenth are wrought iron. Remainder of lead.
> Long Island City and adjacent towns, about one-half are wrought iron.

In Manhattan and Bronx all of the large service pipes are wrought iron, and of the small pipes about half are lead and half are iron. About fifteen years ago the Board of Health allowed the use of wrought iron plumbing throughout, and so in the upper portions of Manhattan and The Bronx many wrought iron service pipes were put in, especially in cheaply built houses.

Absence of the trolley car from the streets of Manhattan has probably deferred the great corrosion of pipes caused by electrolysis, but I am informed that in the northern districts where electric cars have been in use, the effect of electrolysis in corrosion of pipes has appeared, the corrosion and leakage starting at the junction of the dissimilar metals at the corporation cock, where leakage is difficult of detection and repair. The prevalence and constant increase from year to year, of return currents of electricity in the streets underground and following along the pipes, with corrosion by electrolysis at the hidden point where the current leaves the pipe, is the cause of leakage to be most feared in the future, and may warn us not to be too hopeful about stopping all of the underground leakage.

Expert investigation of this matter of electrolysis in its relation to the New York and Brooklyn pipes should be undertaken and steadily maintained.

Past Causes Tending to Decrease Consumption.

Finally, it must be remembered that in New York for years the free use of water has been repressed by low pressure in the mains, that in former years there was nearly always somewhere a scanty pipe system, insufficient pumps, insufficient aqueducts, scant storage reservoirs or gates "gagged off" or severely throttled so that water could not rise to the upper stories of the buildings, and that nowhere in America is so large a proportion of the house-owners compelled to provide house pumps for lifting water to their upper rooms, and that through large sections of the city the $300 "House-pump," entailing an expense for maintenance which doubles the water rates, has stood guard in hastening the repair of defective plumbing in the same way that the $10 water meter hastens repairs in certain cities where a small per capita consumption exists with a high pressure supply.

The question of need of a new supply does not hang upon the subdivisions of the preceding table (No. 2), and if the above subdivisions of use and waste are not just right, *the following main features regarding the consumption remain undisputable:*

The present consumption is 116 gallons per inhabitant per day.

The population is rapidly increasing.

The number of gallons per inhabitant per day used and wasted is increasing steadily.

For four years past the average draft for the year has increased steadily 15 million gallons per day.

With this rate of increase, until additional sources are completed, Manhattan will be close to, or slightly in excess of the safe capacity of its sources, notwithstanding the completion of the New Croton Dam, in 1902.

The proportion of waste is now smaller in New York than in any other American city of half a million or more inhabitants (excepting St. Louis), and, although this great proportion of waste is everywhere recognized, and the fact well known, that the engineering requirements for checking this waste are simple and cheap, *the waste continues in all these larger cities.*

Present Consumption and Waste of Water.

In the year 1899 the consumption throughout the Boroughs of Manhattan and The Bronx averaged..	116 gallons per inhabitant per day.
Out of this 116 gallons, the needless waste averaged at least 50, or more, probably.....	65 gallons per inhabitant per day.
Out of the 116 gallons, the incurable or necessary waste probably did not exceed.....	10 gallons per inhabitant per day.
With a meter on every tap, domestic, public and commercial, and with street mains, service pipes and hydrants thoroughly overhauled and put into as good condition as the average pipe in the streets of Providence or Fall River (all this might take 20 years' time), and with meters frequently tested, to detect under-registry, and with a careful investigation of leaky plumbing and prompt repairs, probably New York would be better served than now and have greater pressure in its pipes with a supply of.....	65 gallons per inhabitant per day.

—than it now has with 116 gallons per inhabitant per day.

The hydraulic and engineering features of preventing the above 65 or 50 gallons of waste are not doubtful or unduly expensive, but would probably require 20 years' broadly planned, thorough work, backed up by strong public support, for their full application.

The real difficulties about saving the water wasted, rest in human nature and are questions of getting the voters to understand the truth of the matter, questions of public sentiment, politics and good government. From studies in the result of efforts at waste prevention in other large cities and from inquiry made among well-informed men as to the present sentiments of the New York public regarding domestic meters, I am led to believe that for the immediate future these difficulties are insurmountable, and that the hope of a cure for any large part of the waste very remote. With all of its present waste, the domestic supply of water costs the average citizen (man, woman and child), of Manhattan and The Bronx, only $1 per capita per year.

Water is so cheap that the average American citizen unfortunately seems better content to buy more than to save or conserve what he now has.

The consumption of water (including waste) per inhabitant is not greater in New York than in the other large American cities. It is, in fact, to-day smaller in New York and Brooklyn than in most other large American cities. There has been a tendency in many recent discussions to over-estimate New York's rank among other cities in wastefulness of water and to over-estimate the saving that can be made because of an apparent failure to appreciate that *the real obstacles to reform are grounded in human nature and not in hydraulic engineering.*

My best guess is that the total leak underground in the streets at present amounts to somewhere between..... 20 and 35 gallons per inhabitant per day.

It can never be known with any degree of accuracy just what this underground leakage in the street is until there is a meter on every tap and a daily estimate made of all public uses and draft for fires; when by subtracting what is measured out from what is measured into the distribution system, the difference will give the leakage from the street pipes. This kind of measurement in Fall River and Woonsocket, with the allowance for a smaller number of joints and of service pipes in New York, gives us the ideal of..... 5 to 10 gallons per inhabitant per day.

—for maximum leakage of street mains and service pipes toward which the water distribution service of Greater New York should work. To attain this ideal will require years of patient work and very heavy expenditure.

Methods Required for Locating the Waste.

To find the leaks in the 1,500 miles of cast-iron main pipes of the city's water works in Manhattan and Brooklyn and to locate among the 250,000 service pipes in these two boroughs those which are leaky; and to find, dig up and plug off the thousands of abandoned service pipes, a part of

which are without doubt leaking badly and all of which are likely to leak sooner or later, is an enormous task and the remedy cannot be applied quickly enough to help out the imminent shortage in the present water supply, but is a task for 20 years' steady, patient, broadly planned work. To find a large leak now or to know where to look for the worst leaks is like hunting for the proverbial needle in the hay-stack. Before we can stop the leaks, we must find them, and no better or more economical method now appears for finding the worst leaks first, than the following—

It is recommended to

1st. Subdivide each borough into districts averaging, say, one-half mile square in Manhattan and Brooklyn, following ward lines where practicable, stop off all water-pipe connections across the boundaries of this district excepting three—feed the water-pipes through two connections at most (the 3d being in reserve and shut ordinarily), and place a Venturi meter on each of the two in use, and thus measure day by day the total quantity of water delivered into each district. Occasionally by the same meters measure the rate of flow at intervals of 5 minutes throughout the 24 hours, to get the relation of the night flow to the day flow and thus determine the total leakage and waste for this one-quarter square mile, much as was done for the whole city in our Central Park experiments. From the records of the forthcoming census, find the population dwelling in each district. From the Water Register's meter records find the total average daily consumption of metered water for all manufacturing and commercial uses within the district.

There will then be available for each district—

The average rate of total consumption and waste per 24 hours.
" " " " night waste per 24 hours.
" " " " total real use per 24 hours.
" " " " manufacturing and commercial use per 24 hours.

From the known population in the district and the character of the houses, a fairly close estimate can be made of the probable real domestic use and this added to the manufacturing and commercial use and deducted from the Venturi meter reading gives the waste—thus checking up with the determination of waste by rate of flow after midnight.

The sum total of all the Venturi meters should be checked up against the recorded delivery of the aqueducts.

Having all this information for each district, those districts in which

the leakage is worst can be picked out and placed first in order for further study.

I have, of course, found no time for working out details, defining the boundaries and estimating the cost of this first step in the location of leakage, but it is plain that it would be comparatively small.

2d. The next proper step would appear to be, beginning in those districts where the greatest waste is found, apply a water meter to every tap not now so provided, public, private and all; keep, with the assistance of Street Department, Fire Department and Sewer Department, an approximate daily record of the amount of water drawn for flushing, fires, etc. (This is found practical and easy in certain other large cities and could doubtless be readily accomplished throughout New York.)

Adding up the sum total of the meter readings in the district for each quarter, the average daily use and waste inside the houses and commercial buildings would be known, and knowing what was measured into the pipes of each district through the Venturi meters, and from this deducting what was measured out through the service meters, plus the draft for fires, flushing and street sprinkling, *there would be had, for the first time in New York, definite knowledge of the amount leaking out through all the leaky joints in main pipes and throughout all of the old, leaky and abandoned service pipes.*

3d. Since to uncover, recaulk, relay any large proportion of the 1,500 miles, more or less, of cast-iron mains in Manhattan, The Bronx and Brooklyn, would be a work of many years with any such tearing up of streets at one time as the public would probably tolerate, and since the immediate relaying of any large proportion of the 250,000 service pipes (more or less) would for similar reasons be inexpedient, it is important to discover and remedy the worst cases of leakage first.

For this is first needed a series of very large scale sectional maps upon which should be compiled all information that records or questions can reveal, as to age and position, size and position of every service pipe, main pipe and gate, both abandoned pipes and active pipes.

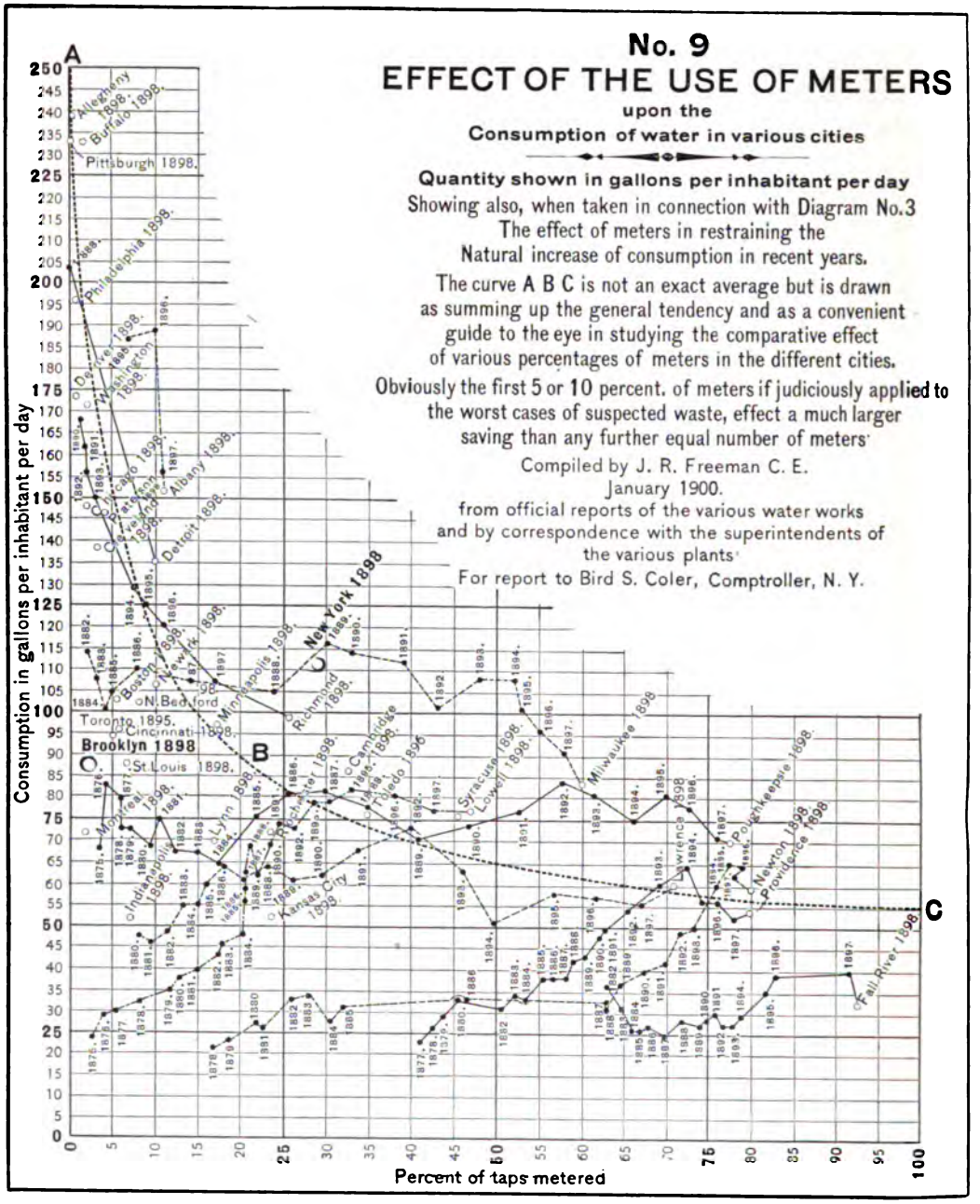
To discover and locate the worst leaks or the street blocks containing them, the districts mentioned in paragraph 1 should temporarily be subdivided into sections of about 1,000 feet square, by closing the water gates, and each section fed through a Deacon meter, and then proceeding by the regular Deacon method, one street after another should be temporarily shut off by night by closing the gates at street intersections, and the effect noted on the leakage diagram. Taken in connection with a special reading of

the service meters through this section, this would plainly indicate which streets should first have the main pipes and services overhauled.

An inspection of the map above recommended in connection with the Deacon Diagram would give grounds for a good guess at the location of the leakage found to exist somewhere within the block.

The absence of stop-cocks on the service pipes within the streets at the curbstone-line prevents carrying the search one step further, as is possible in most cities, but I would very strongly recommend that as pipes are renewed and new services extended, curb stop-cocks, or their equivalent, close to the main *on a combination service pipe for several houses*, be the invariable rule.

It does not appear reasonable to expect that this hunting down, locating and remedying the thousands and probably hundreds of thousands of underground leaks throughout all of New York could be at best accomplished in 10 years, and my present views are that the far-sighted and proper course would be to set such a pace and steadily pursue it, that 20 years hence every main pipe and service pipe about which there can be any suspicion of weakness or leakage would have been overhauled and replaced or made strong enough for a regular working pressure of 100 pounds per square inch, and all fire hydrants meanwhile be brought up to date. This could be readily accomplished with no heavy burden on the water rates if spread over a period of 20 years.


No. 9
EFFECT OF THE USE OF METERS
upon the
Consumption of water in various cities
Quantity shown in gallons per inhabitant per day
Showing also, when taken in connection with Diagram No.3
The effect of meters in restraining the
Natural increase of consumption in recent years.
The curve A B C is not an exact average but is drawn
as summing up the general tendency and as a convenient
guide to the eye in studying the comparative effect
of various percentages of meters in the different cities.
Obviously the first 5 or 10 percent. of meters if judiciously applied to
the worst cases of suspected waste, effect a much larger
saving than any further equal number of meters.
Compiled by J. R. Freeman C. E.
January 1900.
from official reports of the various water works
and by correspondence with the superintendents of
the various plants.
For report to Bird S. Coler, Comptroller, N. Y.
Consumption in gallons per inhabitant per day
Percent of taps metered
A
B
C
Allegheny 1898.
Buffalo 1898.
Pittsburgh 1898.
Philadelphia 1898.
Denver 1898.
Washington 1898.
Chicago 1898.
Albany 1898.
Detroit 1898.
New York 1898
Minneapolis 1898.
Richmond 1898.
Cambridge 1898.
Toledo 1896
Syracuse 1898.
Lowell 1898.
Milwaukee 1898
Lynn 1898.
Rochester 1898.
Kansas City 1896
Lawrence 1898.
Poughkeepsie 1898.
Newton 1898.
Providence 1898.
Fall River 1898.
Toronto 1895.
Cincinnati 1898.
Brooklyn 1898
St Louis 1898.
N. Bedford
Boston 1898.
Cleveland 1898.
Newark 1898.
Montreal 1898.
Indianapolis 1898.

Possible Remedies for Waste.

Water Meters.

After having studied into the conditions as thoroughly as possible in the time at my disposal, I am, from the evidence, forced to conclude that *the one efficient, economical and practical method for lessening the waste of water in New York begins with a water meter on every service pipe.* At the same time I must admit that public sentiment in New York (or the other largest American cities) does not appear to be in favor of domestic meters, but I believe the adverse sentiment comes from lack of full understanding of the benefits to the public welfare and from an exaggerated notion of the inconveniences. There is a growing sentiment in favor of meters in the country at large, and the method of selling water through meters for household purposes, and thus making each man pay only for what he gets instead of forcing him to share in the expense to the city of his neighbor's wastefulness; of placing a premium on economy and a penalty on carelessness and willful waste, has been fully tested and proved entirely satisfactory in Providence, the second city in New England, in Worcester, Fall River, Lawrence and Lowell, the chief industrial cities in Massachusetts, and is meeting with success in Syracuse and Poughkeepsie, New York.

Meters give less annoyance than continued house-to-house inspection.

The experience of various cities demonstrates clearly that there is much more friction between the householder and the Water Department in the method of preventing waste by continued house-to-house inspection by employees looking for leaky plumbing, than when an employee merely enters the cellar for a moment once in three months to read a meter and the householder becomes his own inspector of fixtures, except as he may find occasion to ask for the friendly assistance of an expert inspector to trace some hidden cause for a high meter reading.

The simpler method of a meter in the cellar appears to retain its hold on keeping down waste through the vicissitudes of different administrations much better than the Deacon system with its requirements for special skill of direction and experienced inspectors. The experience of Boston is a case in point. Under the Deacon system, by an exceptionally skillful and vigorous administration, so much waste was stopped in 1883–4 that the consumption per inhabitant was reduced from 91.5 gallons to 68 gallons per day in Boston proper and in the entire district supplied from the Sudbury and Cochituate. As the closeness of inspection has relaxed, and the "new broom" ceased to sweep clean, the consumption has gradually increased

until for the year 1899 it averaged about 120 gallons per capita per day for the same district. Meanwhile, under the system of complete metering, the total consumption in Fall River for all purposes* has in the same period increased only from 32 to 36, and in Providence from 36 to 54. This increase of 18 gallons in Providence is thought by the City Engineer to have come mainly from increased use for manufacturing and for hydraulic elevators worked direct from the city pressure. In Worcester the total consumption, with large use for manufacturing and probably considerable waste from the mains, was only 69 gallons in 1899. In earlier years the total water supplied was not measured.

Citizens and water works officials with whom I have discussed this matter have informed me that the New York public is not educated up to this metering of domestic services and that the New York voters would not submit to it. That this could be overcome by a long campaign of education would appear to be proved by the general satisfaction of the public with meters in Fall River, Providence, Worcester, Woonsocket, Atlanta, and many other municipalities. Personally, I am well satisfied with receiving at my residence in Providence more water for $10 per year through a meter than I received in a suburb of Boston under schedule rating without a meter for $25 per year.

Perhaps past methods and past schedules for the adjustment of water rates are not the best or most popular that can be devised. I am led to believe that if the tariff of meter rates for dwellings is properly adjusted, that when the average house owner comes to clearly understand that domestic meters, to be paid for by the city, will put the burden of waste where it belongs, that a saving to his own pocket can be made by reason of his not having to pay so much extra for water to make good the waste from the undiscovered leaks in street pipes and the leaks in the house of his careless neighbor, and that by the prevention of this waste he will enjoy a greater pressure of water than heretofore, and can meanwhile have every gallon that he now uses, and more, too, at less cost—if these matters can be made plain, it is reasonable to expect that the public will strongly favor metering every tap. To work out the details of an equitable water meter tariff which should give a minimum domestic meter charge of, say, $5 per year for a small tenement, and should for this $5 supply all of the legitimate needs of a family of ordinary size—with subdivisions of meters in tenement houses and apartment houses, much as gas meters are now set for each suite

* The use of water from the public mains by the large factories of Fall River is small. Most of them draw water for steam, washing and water-closets from the river or from ponds.

of rooms. all this would require much study and consultation with Health officers and experts in sanitation to make sure that it was to no landlord's advantage for his tenant to use less than an ample, generous supply, but would not require much more study than is involved in the planning and engineering for a great storage reservoir like that proposed at Cross river.

The greatest cause of house waste is undoubtedly cheap and neglected plumbing fixtures, and among fixtures, tank ball-cocks are doubtless the greatest cause of leakage. The fastening of cocks open, letting the water run unnecessarily is also a great source of waste. Meters tend to a prompt remedy of both of these wastes.

One cause of the smaller consumption in certain English cities is probably the rigid requirements for inspection of plumbing fixtures. In certain of the English cities every fixture must be officially inspected, approved and stamped before a plumber can use it, and this is believed to have greatly lessened the waste.

Brightly polished fittings all look very much alike when new, and considerable loss results in time from those which are light or soft or of cheap design, but with the compression cocks shutting against leather or rubber or fibre discs, common in all American plumbing, it is more often a question of neglect to apply a new disc or washer than of leakage caused by wear or yielding of the metal.

For the underground plug cocks, the corporation cock, or the cellar stop and waste cock, there is a temptation to cheapen the cost of manufacture by adding lead to the mixture of tin and copper or to use the cheaper alloy—soft "yellow brass," consisting only of a mixture of spelter and copper, thus giving a metal which can be worked in the lathe at high speed and with broad cuts.

The experience in those American cities which are fully metered and whose domestic consumption has been already quoted, appears sufficient proof that each householder will become a fairly good inspector of leaky plumbing if he has a water meter in the cellar, and that reputable plumbers will supply their customers with proper fixtures without the inconvenience and expense of a government inspection, if the customer finds it to his pecuniary advantage to prevent waste of water through leaky fixtures (see p. 47).

The great objection urged against meters is the fear that they will tend to a harmful restriction of the use of water in the houses of the poor or in tenement blocks where the landlord may not be of the most liberal class and would try to save water, if metered, at the expense of his tenants' comfort and health. This is a matter which can, I believe, be solved after proper study by some system of separate meters, combined with a carefully considered scale of minimum charges.*

* Perhaps the inventor can help solve this problem *for those who fear injurious restrictions of use* by the presence of a meter, by a meter register with an extra visible counter like that of the improved bicycle cyclometers which have one counter that can be set back to zero at the beginning of a run, so that the tenant can keep the run of the use. But practical experience in the metered cities appears to prove that this is an unnecessary complication for the average consumer.

The reliable answer is found in the test of experience. *No complaint, so far as I am aware, now exists in Fall River, Providence, Worcester, Lawrence, Syracuse or Poughkeepsie, Atlanta or Richmond, that the comfort or health of any class in the community suffers because of the application of water meters.*

City Ownership of Meters and Service Pipes.

For the best results in saving waste these house meters and the entire length of all service pipes, from the main in the middle of the street to inside the cellar wall, should be paid for and put in by the city, and the city be responsible for their good repair, and the meters should be periodically tested free of expense to the householder. I am of the further opinion that the city, at the same time and at its own expense, should apply a curb stop-cock to every tap.

Systematic work in this direction should at once be begun, looking toward the city examining and replacing all service pipes of whose tightness there is any doubt, at some fixed rate of progress per year, looking to the entire overhauling and remodeling of these within a period of, say, 20 years, relaying them with fewer main arteries under the roadway, branching near the curb into the several buildings and with a curb stop-cock on the pipe into every building, at the same time making all new pipe thick and strong enough to stand a future working pressure of 100 pounds per square inch. Taking the work up in this manner, systematically, with the aim and end continually kept in view, of having all doubtful pipes relaid and made good within 20 years, would not burden the water debt revenues seriously and the 20 years would quickly pass.

These curb stop-cocks are now the rule in almost every city excepting New York, and one or two cities have cut them in subsequently to the original construction. They are of the utmost convenience and utility for detecting waste. In quantity, under ordinary conditions, they can be put in for from $5 to $6 each for the ordinary sizes.

In the most crowded portions of New York there are difficulties arising from the general use of vaults under the sidewalk and from the tearing up and patching of asphalt or other pavement that would add to the expense. A large portion of Boston was originally piped without them, but they were subsequently added. About 6,000 cut in in 1887 averaged costing about $5 each for all labor and material. Paterson, N. J., is another city where it has been found profitable to add them. Within the past two years, under the efficient administration of J. Waldo Smith, C. E., Superintendent of Water Works, about 2,000 curb-stops have been set at prices averaging not far from $5 to $5.50 for the ordinary domestic sizes, as one effective means for aiding in the restriction of waste.

It appears that this work of providing every tap with a curb-stop might without inconvenience proceed at a rate which would in ten years equip every tap in Greater New York not now so provided.

Expense of Applying Meters and Time Required.

By taking hold of this matter of introducing meters earnestly, every tap in New York and Brooklyn could be provided with a meter within five years from the time that the approval of the public was assured.

The average cost of cutting in a water meter located in the cellar upon an old service pipe, including all labor, pipe fitting and a support or box for the meter, as found by the average of a large number of jobs upon the common domestic sizes of service pipe, mainly ⅝ inch and ¾ inch, under the different practice and methods of fitting up of the different cities, is stated to me by water works officials in these cities to average as follows :

Lawrence, Mass.	$1.50
Malden, Mass.	1.47 to $1.66
Providence, R. I.	3.75
Paterson, N. J.	3.10

The cost of applying meters in New York and Brooklyn with the work done by the city, working systematically on a large scale, for the labor and material for setting the meters, would probably average not above	$2.50 each.
Meters of the common domestic sizes, partly 5-8, many 3-4, and a few larger, at a fair average price, would cost	12.50 each.
Average cost per tap of metering would probably not exceed.	$15.00

There are said to be at present about 86,000 unmetered taps in Manhattan and in The Bronx. Therefore, to meter every unmetered tap in Manhattan and The Bronx would cost not far from, 86,000 × $15.00 = $1,290,000; or, working under good organization on a large scale, the work could be done for less.

In other words, to meter every tap now remaining unmetered in Manhattan and The Bronx would probably cost less than the land, legal expenses, damages and cost of construction for the proposed new reservoir in the Croton Watershed at Cross river, and would probably give two or three times as much water, and would furthermore permit the pipe-waste to be systematically located so that a still further saving could be made.

Probable Saving in Manhattan by Meters.

The saving by meters in preventing waste from poor plumbing, and careless or willful waste *inside the house*, apart from the saving that can be made by investigating and stopping the leaks *outside the cellar wall*, after the system got well established, would, as I believe, judging by experience with meters in other cities and by the measure of the total waste which now cer-

tainly exists, very soon amount to 15 or 20 gallons per inhabitant per day, and possibly to 25, and after 10 or 20 years of education in economy of water and the natural gradual weeding out of poor fixtures the saving, as compared with the present waste, might reach 30 or 40 gallons per capita per day.

A saving of only 20 gallons per capita daily on 2,000,000 population, would make a total saving of 40 million gallons per twenty-four hours.

While by the reservoir proposed at Cross river to hold 9 billion gallons of storage, the addition to water resources of the city in a time of extreme low rainfall probably would be only 12 million gallons per day.

Measured from another point of view, the saving by domestic meters would probably be equivalent to supplying from 3 to 5 years' natural increase of consumption at the decreased rate, but it might take a long time to obtain the full extent of this saving and to overcome the wasteful habits that had been long established and to discover and repair the many leaks inside the houses.

No such economy could be hoped for at first as that obtained in the cities that, like Fall River, have been fully metered all the time since the works were built, and where the house owners and plumbers have been thus educated, that good fixtures are most economical although costing a little more at first.

Other Methods than Meters for Preventing Waste.

So much has recently been said and published in the daily papers concerning other methods for preventing waste in New York City, that it is best not to dismiss this subject without a few words about the other *practical* methods.

The water waste prevention problem has engaged the earnest attention of water boards and engineers in more than a hundred cities for the past twenty years, and the writer believes facts justify the statement, speaking broadly, that out of all this experience, only three successful methods of preventing waste in a large community have been devised.

The first is that of a meter on substantially every tap, already described.

The second is that of house-to-house inspection for leaky plumbing.

The third system is that known as the Deacon system of water waste detection.

Cost of Maintenance of Water Meters.

On page 75 I have estimated the average cost per tap for metering all of the service pipes not now metered in New York, these consisting mainly of domestic services at $15 each, including all material and labor.

The question next comes as to the cost of the yearly maintenance of these meters, as an important fact to be known before deciding how much the net saving by meters really is, and whether it is cheaper to waste the water which meters undoubtedly save than to meet the large expense of introducing and maintaining meters.

This question is so important that an extended analysis of the experience in various largely metered cities would be well worth its trouble and expense. These would be considerable, for the records will seldom be found so separated and classified as to give just the information wanted without much working over.

Life of a Water Meter.

I have not had time to give this problem the attention it deserves, but have personally made some brief examination of the records of the Providence, R. I., Department of Public Works, which has for many years maintained the largest number of meters to be found in any one American city excepting New York, and has been using meters extensively for more than 25 years. In 1880, twenty years ago, Providence had 45 per cent. of all its taps metred and 4,401 meters in use.

On Sept. 30, 1898, Providence had	16,246 meters.	In 1890 had 9,286 meters.
On Dec. 31, 1898, Fall River had	6,128 meters.	In 1889 had 3,138 meters.*
On Nov. 30, 1898, Worcester had	11,878 meters.	In 1889 had 7,354 meters.*
In 1898, Rochester had	7,227 meters.	In 1889 had 1,800 meters.*
On June 30, 1898, Syracuse had	5,806 meters.	In 1889 had 430 meters.*
In 1895, Milwaukee had	15,951 meters.	In 1889 had 30 meters.*

The Water Department of Providence has from the first been under excellent systematic management and has maintained excellent records. Some excellent notes on its experience in cost of maintaining meters, by Edmund B. Weston, C. E., will be found in "Journal Engineering Societies, 1888," page 303. I have not had time to secure a complete analysis of these records for the 14 years that have elapsed since this analysis by Mr. Weston, but from a brief examination of these records, very courteously afforded me, and from conference with Mr. Otis F. Clapp, City Engineer, and Mr. Austin, Clerk of the Meter Department, the following facts are established:

The average life of the ordinary domestic meter, of a good type, well cared for and with occasional repairs and renewal of worn parts, may be safely assumed at not far from 20 years.

There are very many meters from 20 to 25 years old still in use in Providence giving excellent service. Out of the 391 meters condemned for all causes, including freezing, during 1899—which is 4 per cent. of the number in use 10 years ago—a cursory examination of the records shows many to have been in use for 25 years and a majority more than 20 years, and very few less than 15 years. Probably Providence has been more careful than most cities in its selection of meters, but with the experience now obtained any city should be able to do as well in the future.

Annual Cost of Repairs.

The number of water meters in use in Providence, December 31, 1899, was.....	17,124
" " 31, 1898, "	16,388
Increase in total number during year.....	736
Number of old meters replaced by new during year.....	391
Total number of meters set during year.....	1,127
Average number of meters in use during year.....	16,756

* Per "Manual of American Water Works," 1879–1880.

To set these 1,127 new meters and to care for the average number of meters in use during the year required the services of

1 Foreman Machinist @ $1,100 —		$1,100 00	per year.
2 Machinists @ $1,000 —		2,000 00	"
1 Assistant Mechanic @ $780 —		780 00	"
Total 4 men		$3,880 00	"
2 one-horse teams, which we assume cost, all told		800 00	"
The total sum charged to repairs of meters was	$6,835 53		
Of which above wages amounted to about	3,880 00		
Leaving as approximate cost of repair parts and supplies	$2,955 53	2,956 00	"
I assume as fair allowance for use of quarters and tools, etc., say		1,000 00	"
Approximate annual cost setting 1,127 new meters, caring for 16,756 meters in use, the average meter in use being, say roughly, now 11 years old.		$8,636 00	"
To get at cost of caring for and repairing old meters. We should properly deduct actual cost for setting up the 736 new meters. I have not had time to make a close estimate on this from time books, but it must have been at least, say		1,000 00	"
Leaving an annual cost of maintaining 16,756 meters averaging over 10 years old		$7,636 00	

In a large system of meters, once in permanent condition with renewals all the time going on, with an average life of 20 years, the average meter will be about 10 years old.

Average cost in Providence, R. I., of repair and maintenance per meter per year $\frac{7634}{16756}$ = 46 cents.

The figure above, deduced somewhat roughly from recent experience, is found to agree remarkably with the figure of 44 cents deduced by Mr. Weston from the Providence experience for the six years 1878 to 1886; the fact that the average meter is now older being perhaps offset by the use of a more durable type of meter than some of the types at first used.

Cost of Reading Meters and of Computing Meter Rates.

To read the meter registers quarterly, and to compute and make out the bills for the average number, 16,756 meters, in use in Providence in 1899 required the work of 5 men and a superintendent, but in between times these men did much other work, such as inspecting the fixtures in the comparatively few unmetered buildings, helping in delivery of water notices and occasional assistance to the service clerk in turning water on and off.

The total of the salaries for these six men for the year was about	$6,000 00
There is no team employed on this work, but city furnishes street-car fare—perhaps	200 00
I will assume as a fair allowance for cost of quarters and stationery	800 00
Total cost for reading meters, etc.	$7,000 00

Making no deduction because of the other work done by these men the average cost per meter per year for 4 readings is $\frac{7000}{16756}$ = 42 cents.

Summary of Cost of Metering Water on a Large Scale, Well Organized.

Average Expenses per Meter per Year.	Actual Experience in Providence, Rhode Island, about).	Assumed for Estimate for New York.
Interest on cost of meter and setting (3% on $15 for N. Y.)	50 cents.	45 cents.
Depreciation and renewal of meter. Life assumed 20 years.	75 "	75 "
Maintenance and repairs, and testing and resetting	46 "	70 "
Reading water registers and computing bills	42 "	60 "
Total cost per meter, per year	$2 13	$2 50

For Manhattan and The Bronx—The additional expense for the 86,000 domestic services not now metered would, per the above, be 86,000 × $2.50 — $215,000 per year.

Which, judging by recent Boston experience with the Deacon meter, is much less than it would cost to maintain a waste water detection service of that kind, with its far lower efficiency.

I have little doubt that the complete introduction of meters, paid for by the City, the City assuming also the ownership and paying for the meters already in use at their fair present value, and thus leading to a total yearly expense for interest, maintenance, repairs and renewals of, say, 120,000 × $2.50 — $300,000 per year for Manhattan and The Bronx (a large part of which is now expended on present meter service and waste detection), would save of house waste within a very few years 40 million gallons of water per day, which at the low value of $25 per million gallons would be worth 40 million gallons × 365 days × $25 — $365,000 per year.

And believe this would quickly lead to a much larger saving by making it practicable to locate more of the waste from street mains and service pipes than can be done without them.

On the other hand, since, as elsewhere explained, a large additional supply appears imperative, there is no need to incur both expenditures for meters and for new supply complete all at once.

The new supply (up to at least 150,000,000 gallons) will probably be had almost as cheaply in full as in part and the two reasons for encouraging meters are:

First—To conserve the present sources and avoid danger in the years that must elapse before the new source is ready.

Second—The gradual education of the public in matters of water meters toward the not very distant day when substantially all service pipes must be metered.

Third—As a part of the necessary progress toward so reducing the bulk of water delivered, that filtration of the entire Croton and Bronx supplies may come within bounds of practicability.

Therefore, while metering of all taps should be encouraged, it need not be forced for 5 or 10 years to come, providing the Housatonic source can be obtained.

House-to-House Inspection for Leaks.

The second system, that of house to house inspection requires, in order to be at all effective, that there be a curb stop-cock, and these do not exist in Manhattan to any noteworthy extent. The method of house-to-house inspection which causes least annoyance to the householder and least expense to the city for inspectors and detects the most leakage, is substantially as follows:

1st. A night inspection is made, mainly between midnight and four A. M., by an inspector going from house to house armed with a long key or wrench for turning the curb stop-cock. The upper end of this wrench may have a "waterphone" attached, consisting of a small resonant chamber and a cup to fit against the ear, or the end may be simply flattened and the ear held against it, or the inspector will soon become expert in hearing the noise transmitted through the key by simply placing the ordinary end of the key in the ear.

By listening with the key on the cock any large flow of water is made manifest by the sizzling noise transmitted; and if no noise is heard the cock is partly closed, or for still smaller leaks entirely closed for a moment and then opened slowly. If no noise of leakage is heard it is assumed that the house plumbing is free of leakage and the householder saved the annoyance of an inspection of fixtures the next day. On the other hand if leakage is heard a record is made for the day inspector. The time taken at each house by this night inspection is not long, and a fair idea is obtained as to the houses in which there is leakage, although not much idea can be had of its magnitude.

The day inspector goes to every fixture in the suspected houses searching for leaky fixtures or leaky pipes, and notice is served requesting prompt repair.

The Church waste-detector stop-cock, devised by the former Chief Engineer of the Croton Aqueduct, and introduced to a limited extent in New York some years ago, was merely a curb stop-cock adapted to give a closer estimate of waste than that obtained by placing the ear against the ordinary pipe-wrench, and was merely an instrument for use in this method of house to house inspection.

The Deacon System of Water Waste Detection.

This was introduced by Mr. George F. Deacon, Chief Engineer of Water Works in Liverpool, England, about twenty-five years ago, and extensively used in parts of Boston, Mass., for more than fifteen years. The methods and the results of its use in Boston are described in a paper by Dexter Brackett, C. E., before the Boston Society of Civil Engineers, May 17, 1882, published in the "Journal of Association of Engineering Societies." The results are also stated in the report of the Boston Water Board for 1884. The method has been tested to a very small extent in Philadelphia. This method is still very successfully followed in Liverpool, Glasgow and certain other European cities.

This Deacon System is about to be abandoned in Boston because, as I am told, it has recently cost about $100,000 per year to maintain this service for 190 districts, containing about 350,000 population, and it is thought cheaper, under present conditions of an abundant supply, to let whatever waste this saves go. In the Boston Deacon Waste Detection Service 84 meters were in use last year. The force engaged in operating the meters consisted of 48 men, with 20 more men engaged on the house-to-house inspection branch of this work, a total of 68 men. One meter in some cases serves for as many as 4 sections. The cost of the meters was not far from $250 each and the cost of setting probably not far from $150, a total of $400 per meter.

The cost of operation, of course, depends largely on the methods adopted.

The cost of maintaining mechanically is small.

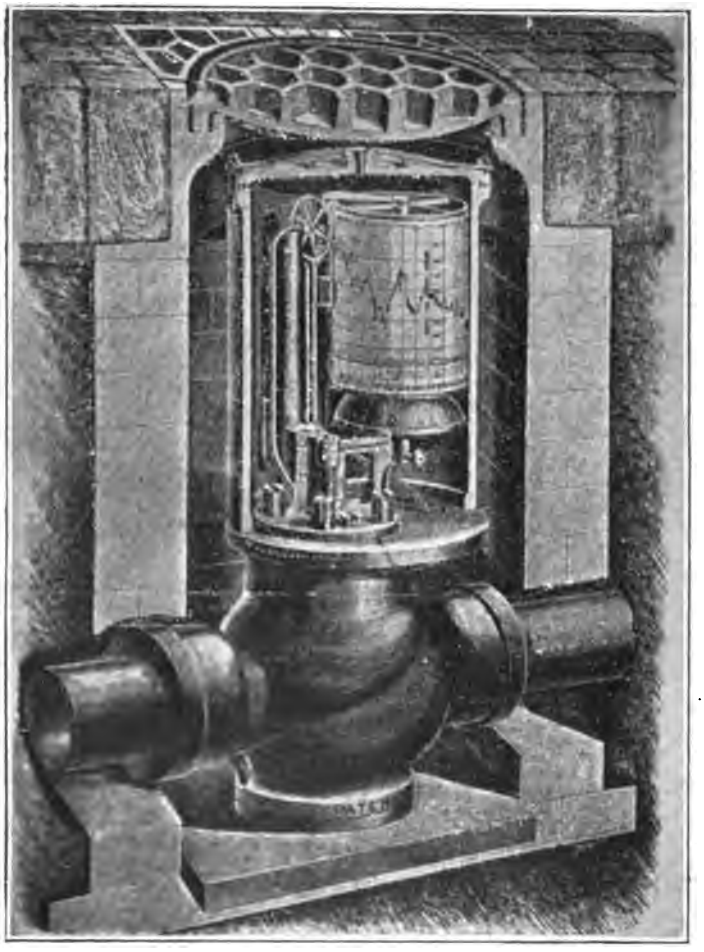


Fig. 10. DEACON METER.

The meter is most conveniently located on a by-pass in a pit under or near the sidewalk with gates so arranged that the meter can be thrown in or out of the circuit.

The size found most convenient in Boston had 6-inch inlet and outlet pipes.

This size would commonly serve for measuring the flow in a street main up to 12 inches diameter.

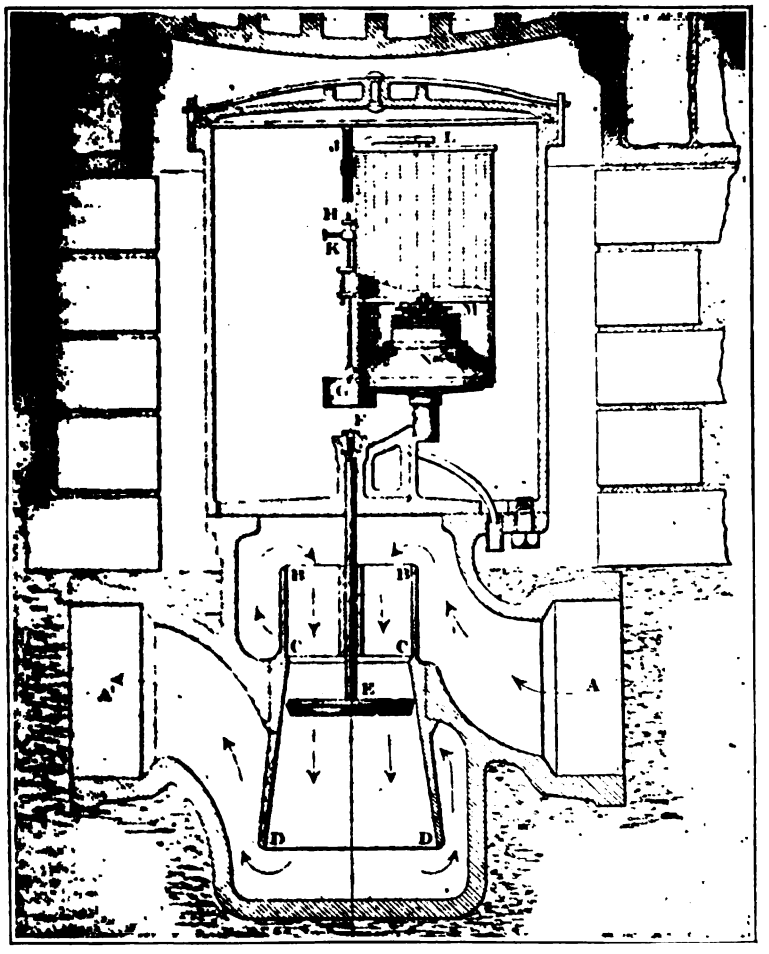


Fig. 11. DEACON METER.

The principle of operation is that of an orifice of varying size discharging under a constant head, and with the motion of the piston E, which controls the area of the orifice, and therefore is proportional to the rate of flow in gallons per minute, recorded upon a diagram carried by a cylinder revolved by clockwork.

The water enters at A, B, C, forces down the piston E, which is counterbalanced by the weight G suspended by the wire F, carrying the pencil K and passing over the pulley J. The taper of the brass hollow cone C, D, graduates the area in proportion to the travel of the pistons.

The following brief description in connection with Figures 10 and 11 will make the method plain :

The system of distributing pipes is subdivided into a large number of districts, each very much smaller than the limits of the smallest New York City ward. These subdivisions in the Charlestown District of Boston averaged about 1,000 feet square.

During the test the district under examination is fed through only the one connection on which the Deacon meter is attached, so that every gallon of water going into the district is metered by a meter which, unlike the ordinary water meter, does not record the total number of gallons that goes through, but instead, records by a line on a strip of paper, *the rate of flow* at every moment, day and night. Then, by studying the relation of the draft of water in gallons per minute after midnight and until 4 o'clock A. M., when domestic and factory use should be almost nothing, to the draft in the hours of daylight and activity, an opinion may be formed as to the proportion and amount of the whole draft that is wasted. Next, an effort is made to locate this waste by three sets of operation.

First—By a large corps of inspectors working at night, going from house to house, stopping at each tap and listening with the ear against a waterphone or rod leading to the sidewalk stop-cock, for the sizzling noise that accompanies the passage of water through the partly closed cock. If no noise is heard with the cock wide open, it is partly closed, and the passing of a very small quantity of water can thus be heard. The service pipes where leaks are found are closed one after another, and a note is made of the house numbers where such leaks or drafts are found, to be used by the day inspector, who will examine the house fixtures and plumbing on the following day. Toward morning, or about 4 A. M., the night inspector retraces his steps and opens the house service cocks that he has closed earlier in the night. A record of the time of opening and closing each service cock is made, and meanwhile the Deacon meter is registering on a strip of paper the rate of flow at each moment, and so shows a change corresponding to the shutting off or opening of any large leak.

Second—The day inspector takes the night inspector's records and goes to each of the houses where the sizzling in the service cock had indicated a leak and makes an inspection of each sink tap and water closet fixture and bathroom fixture within the suspected house, or explores along the entire circuit of the house-pipes, to determine the amount of leakage and enforce a remedy.

Third—To detect the leaks in the service pipes between the street main and the cellar, the street-crossing water-gates are shut at night so as to shut off the water first from one street and then from another, until all streets in the district are shut off. The special recording meter meanwhile continually registering the change in rate of draft. Another operation for measuring the leaks in the main pipes is to about midnight shut off all the house service pipes along a street or within a district, when, of course, the flow shown by the registering Deacon meter for that street can only be accounted for by leaks in main pipes and service pipes.

In the climate of Boston this method had to be suspended during the winter months, and the same would be true in New York.

By this method the consumption in Liverpool was greatly reduced and brought within the capacity of the works, so that pipes were enabled to be kept full twenty-four hours a day instead of having to be shut off fourteen and one-half hours out of the twenty-four to check consumption, as was the case before the inspections were introduced ; and in Boston as already stated, by the use of about seventy of these meters, the consumption in the city proper and the suburban districts supplied from the Cochituate and Sudbury works was in 1883–4 reduced from 91.5 to 68 gallons per inhabitant per day.

This Deacon system was temporarily abandoned in Boston by the city authorities about 1895, mainly perhaps because of saving the wages of the large corps of inspectors, and because the opening of the new Metropolitan

supply makes the burden of waste less apparent, but was subsequently resumed, but, as I am informed, is about to be abandoned again because of its cost being out of proportion to the value of the water saved. For the year 1899, with a Deacon Waste Detection Service costing $100,000 for the year, Boston averaged using 120 gallons per capita, or a little more than New York.*

I have been led to conclude, and I believe that any competent hydraulic engineer who investigates the matter closely will conclude, that *the Deacon system is not so economical and so well adapted to New York as the American system of individual meters* followed in Fall River, Providence, Worcester, Woonsocket, Poughkeepsie and many other places, and which I have so strongly recommended for New York. I am told by Mr. Birdsall, Chief Engineer, Water Department, that a Deacon meter was imported many years ago and given some trial, but found inapplicable to New York conditions.

In applying the Deacon system to New York, the first difficulty is that *New York's house service pipes are nearly all laid without curb stop-cocks*, a faulty method rarely found in other cities. In New York (Manhattan) I am told by Mr. Birdsall that the only stop-cock between the cast-iron main and the cellar is the " Corporation Cock," driven as a hollow plug without a screw-thread into the cast-iron main, and that, contrary to almost universal custom in other cities, the New York house-owner, at his own expense and by his own plumber, puts in the service pipe. The corporation cock is buried and wholly inaccessible (except by tearing up the paving and digging down, which, of course, is inadmissible), and thus the *primary inspection of the Deacon system is impossible in Manhattan*, if not throughout New York.

The second difficulty in using the Deacon system in New York is the great difficulty of securing the large number of competent night inspectors and competent supervisors, for the method involves much clear-headed reasoning and exactness of manipulation to quickly locate small leaks. In house-to-house inspection, I am told by superintendents who have tried it, that it takes six months to educate a new inspector, such as ordinary pay attracts, into becoming at all expert in quickly discovering small leaks.

The third difficulty is that the average citizen would not take kindly to the frequent and continual intrusion into the privacy of his home of unknown and it may be irresponsible inspectors, free to range from cellar to attic, hunting for leaky fixtures.

* At the same rate of cost per capita the Deacon Meter Service for New York and Brooklyn would cost nearly a million dollars a year. The management is probably as likely to become extravagant and inefficient in New York as in Boston.

House to House Inspection in St. Louis.

I have elsewhere in this report noted St. Louis as the one large American city where the per capita consumption is slightly smaller than in New York, being in 1898 about 88 gallons per capita. I have not had time to examine into and analyze the causes which make it so, but from such little personal knowledge as I have of the conditions and of the men in charge, am inclined to view it as in part at least the result of good supervision, but in part perhaps to the different conditions of supplying some of the cheaper houses from a "hydrant" in the yard, and possibly to some different basis of counting population supplied.

A very instructive paper describing the "house to house inspection to prevent waste" as practiced in St. Louis in 1884, read before the Engineers' Club of that city, by Mr. M. L. Holman, is to be found in Journal of the Association of Engineering Societies, August 1885, page 368.

In January, 1884, the waste through leaving faucets open to prevent freezing became so large that the distributing reservoir was emptied and the pumps could not supply the demand. Heroic treatment became imperative, and a water waste detective service with 40 inspectors was promptly organized, and search for leaks carried on with great vigor for a few months. Briefly, the method of search for night waste was to place a long valve key firmly on the curb-stop (curb-stops are missing in Manhattan), place the ear against the flattened top of the key, partly close the cock and listen for the sizzling sound which accompanies flow through a partially closed cock. By day the house plumbing was also inspected, accompanied by the use of the curb-stop key as a "waterphone."

As a sample Mr. Holman gives inspections of one night, January 31, 1884. Of about 100 houses inspected 66 per cent. were all right, 34 per cent. were wasting water.

The day gang following up the night gang by inspecting the plumbing found that of the 34 per cent. of houses having night waste 13 per cent. of entire number had plumbing all right, 21 per cent. of entire number had plumbing in bad order.

To give an idea of the permanence of "reform," Mr. Holman carefully re-inspected a district which had previously been twice gone over and the water shut off from those who had not promptly complied with the order to reduce waste. Nevertheless, the re-inspection showed that *about half of the houses in the district were again wasting water.* The temporary saving during this period of freezing weather was apparently not far from 12 gallons per inhabitant per day.

Mr. Holman reports that although this work was carried on for only 2 months, it became very burdensome to householders, and gives as his opinion that the great saving made at first by this method is not permanent.

Mr. Thomas J. Whitman, C. E., the experienced water commissioner of St. Louis, also reports that he doubted the permanent value of this system of night and day, house to house inspection.

Waste Inspection in Cincinnati.

Cincinnati is another city whose experience in waste prevention without the use of complete metering is instructive.

Between November, 1881, and July, 1882, the systematic search for waste by means of the "waterphone," an instrument which in its best and simplest form may be in general described as a long wrench for turning the curb stop-cock, having a suitable extension at the top against which the ear can be placed. From page 29 of Mr. H. C. Meyers' compilation of data on waste water prevention, it appears that the consumption was cut down to about 17 gallons per capita less in 1882 than in 1881. It was concluded, after allowing for some increase in legitimate consumption, that the reduction in waste amounted to about 21 gallons per capita.

Again, as showing the lack of permanent improvement in saving waste by house to house inspection after the new broom has ceased to sweep clean, it is of interest to examine the curve of consumption for Cincinnati on Diagram No. 4, and to note how, after sharply dropping 20 gallons per inhabitant per day from 1881 to 1884, it steadily and rapidly rose to 135 gallons per inhabitant per day in 1895 and was thenceforth held down by lack of pumping engines.

It is also of interest to note that the experienced engineers who reported upon an increased supply for Cincinnati in 1896, base their estimates for extensions upon a consumption of 130 gallons per capita per day, and state, page 5:

"A careful consideration of the question of per capita daily consumption indicates that with the general introduction of meters into the city during the coming 40 years, the average daily per capita rate should not exceed 130 gallons."

What Quantity is Needed for Future Supply.

The quantity of water needed in the future depends largely upon the measures taken to prevent waste. I am assured by various gentlemen familiar with public opinion and the course of public matters in New York City, that the attachment of a meter on every service pipe is contrary to the general desire, and therefore impossible of attainment. Without this universal metering no successful method of permanently stopping more than a fourth part of the probable leakage appears. To find the proverbial needle in the haystack is just about as easy as to find the leaks in Manhattan's eight hundred miles of mains and its six hundred miles, more or less, of service pipes, without some separation like that suggested on page 68.

Without a meter on nearly every tap, in my judgment, the only safe course is to provide in New York for a consumption of about 150 gallons per inhabitant per day, and to expect the amount to increase from year to year until, say, twenty-five years hence the consumption would reach 200 gallons per inhabitant, notwithstanding that factories, stores and business houses are all metered and that as great diligence as is found in the average American city is used in trying to discover leaks and prevent waste. With the increasing use of electricity for railways and other purposes, electrolysis of pipes is almost certain to increase the waste unless the matter is very carefully watched.

In providing for the health, comfort and business prosperity and the convenience of the community, we must be governed by the practical condition of what the public will do, and not by our theory of what they ought to do. *We must provide water enough so that under existing conditions, with things and men as we are to find them, no man shall suffer from thirst or lack of water for washing* by reason of his neighbor's waste, and we must deprive no steam boiler of its water simply because of the inability to relay some hundreds of miles of leaky service pipes and leaky mains.

Even with an active start toward complete meters, looking at it from the practical rather than the ideal side, I see no hope whatever of avoiding or of deferring a large new supply. The present conditions demand it with all possible promptness.

If the Ten Mile and the Housatonic can be obtained, the water must be taken nearly all at once; and then it would be real economy to defer complete meters for perhaps 10 or 15 years. But I believe that ultimately complete metering must surely come.

QUANTITY PER INHABITANT THAT SHOULD BE PROVIDED FOR IN FUTURE.

While it is probable that with a meter on every tap, and with ideal management after a thorough investigation and remedy for leaks in service pipes and mains, and a thorough reorganization of methods, the consumption of all boroughs of Greater New York could be brought down to somewhere between 50 and 75 gallons per inhabitant per day, and meanwhile afford a full and generous supply to every householder, to all manufacturers and to all commercial interests, I do not think it probable *under the conditions which exist in New York and the other large American cities*, that by the most strenuous efforts in locating and repairing leaky mains and by the attempted application of meters to every service pipe, the consumption can be brought below 100 gallons per inhabitant per day and held there, and believe it more likely to be 125 gallons per day ten years hence, however much the question of waste prevention be agitated now. It will endanger the public safety to be too hopeful about the prevention of waste and therefore defer the construction of new works.

The total cost of water for domestic purposes to each head of a family in New York is now only $1 per inhabitant per year, or considerably less than $10 per household. The average American citizen in New York or elsewhere, ignorant or distrustful of the fact that a meter in the cellar causes him no trouble, and that it, together with a well-organized administration, would save him a part of his water bill without even a thought of curtailing the most lavish use, appears well satisfied to pay $10 where he might pay $5, or to let the city spend another twenty-five millions or more within the next five years, instead of deferring this expenditure fifteen or twenty years, and meanwhile he contents himself with growling a little when the pressure falls off one or two stories because of overloading the pipes or overloading the pumps to supply waste.

OUTLOOK FOR PREVENTING WASTE LESS HOPEFUL IN THE LARGE CITIES.

In some of the smaller cities, public interest has become quickened in these matters, but not in any of the large cities, as will be clearly seen by inspecting Diagram No. 4 and the following table, or Diagram No. 9. Fall River, Providence, Worcester, Woonsocket and some other smaller places were very largely metered from the start, and the public is very well satisfied with the method. The actual consumption in various completely metered cities of size large enough to be fairly comparable in water use with the metropolis is given on page 47.

Turning to the larger cities where conditions of administration are more difficult, among the many examples of Table No. 1 and Diagrams Nos. 4 and 9, we find, in the case of Boston, that notwithstanding it had established eighteen years ago the most elaborate inspection and water waste detection service that has ever yet been introduced into an American city (other than the system of complete metering established in Fall River, Providence and Worcester), and succeeded for a time in reducing waste to the extent of about 21 gallons per inhabitant per day, down to 71 gallons per inhabitant, the consumption has since increased with relaxation of the thoroughness of inspection, until during the year 1899 it averaged nearly 120 gallons per inhabitant per day for Boston and those outlying districts formerly supplied from the Cochituate and Sudbury works.

In planning the Boston Metropolitan Supply about six years ago, a very careful investigation was made as to the probable requirement by Mr. Dexter Brackett, now Engineer of the Distribution Department of the Metropolitan Water Board, and for many years the assistant to the City Engineer, particularly charged with matters relating to the water supply, and who probably has a richer fund of experience in water waste prevention to draw upon than any other man in the United States.

He estimated as a suitable allowance for the Boston Metropolitan District for the next thirty years:

For domestic use	35	gallons	per	capita.
For trade and mechanical purposes	35	"	"	"
Public use	5	"	"	"
Waste	25	"	"	"
Total	100	"	"	"

During the year just closed the use has averaged 110 gallons for the entire district supplied, including much suburban territory, due without doubt mainly to lack of restriction of waste.

In Philadelphia the Commission of eminent engineers, Mr. Rudolph Hering, Mr. Joseph M. Wilson and Mr. Samuel M. Gray, appointed by Mayor Ashbridge in 1899 to investigate the improvement of the water supply, *after an earnest recommendation for meters, based their estimates upon a daily requirement of* 150 gallons per inhabitant per day.

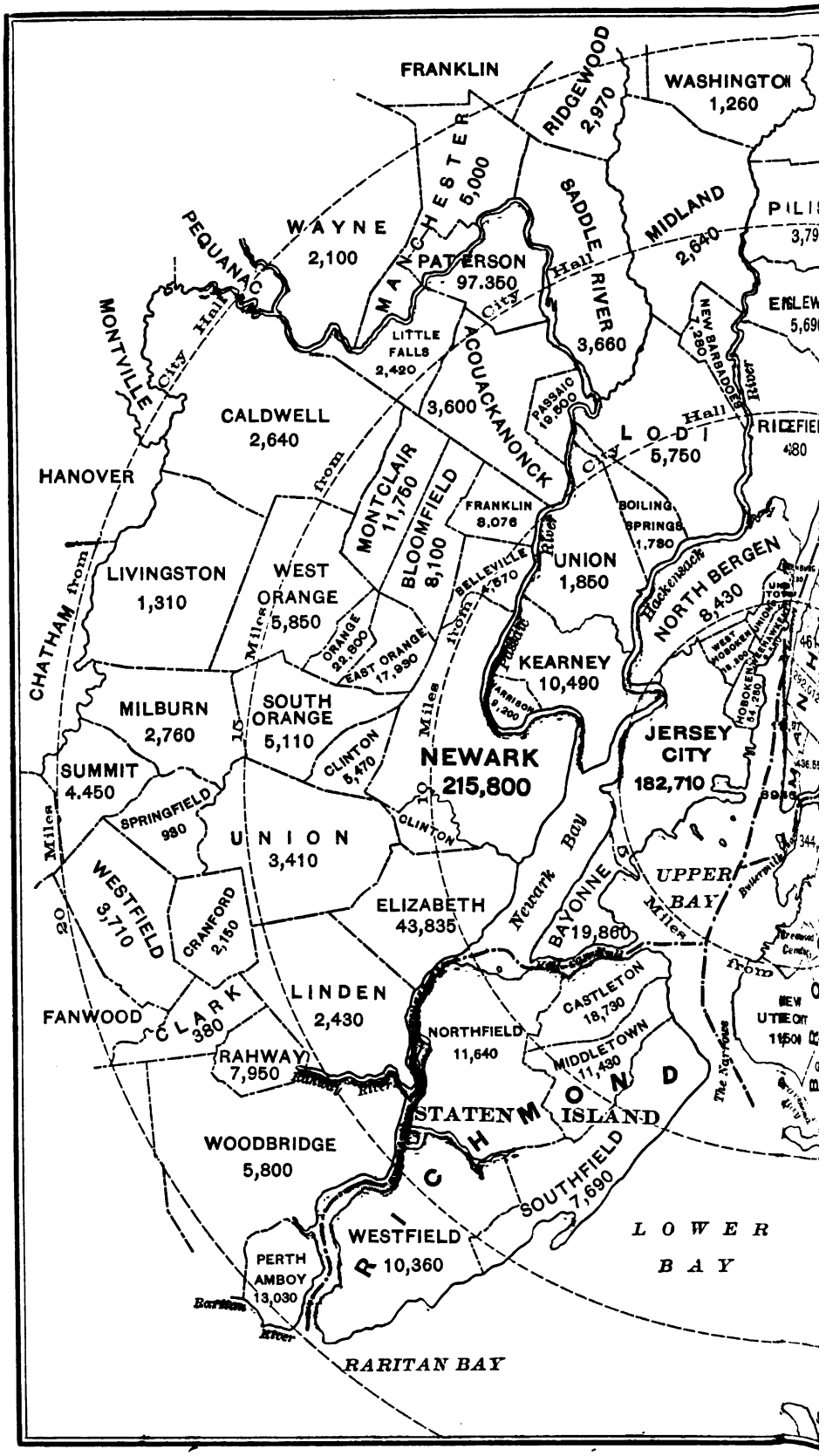
The present consumption in Philadelphia was estimated by them at about 169 gallons per inhabitant per day, of which it is well understood that a very large proportion is waste.

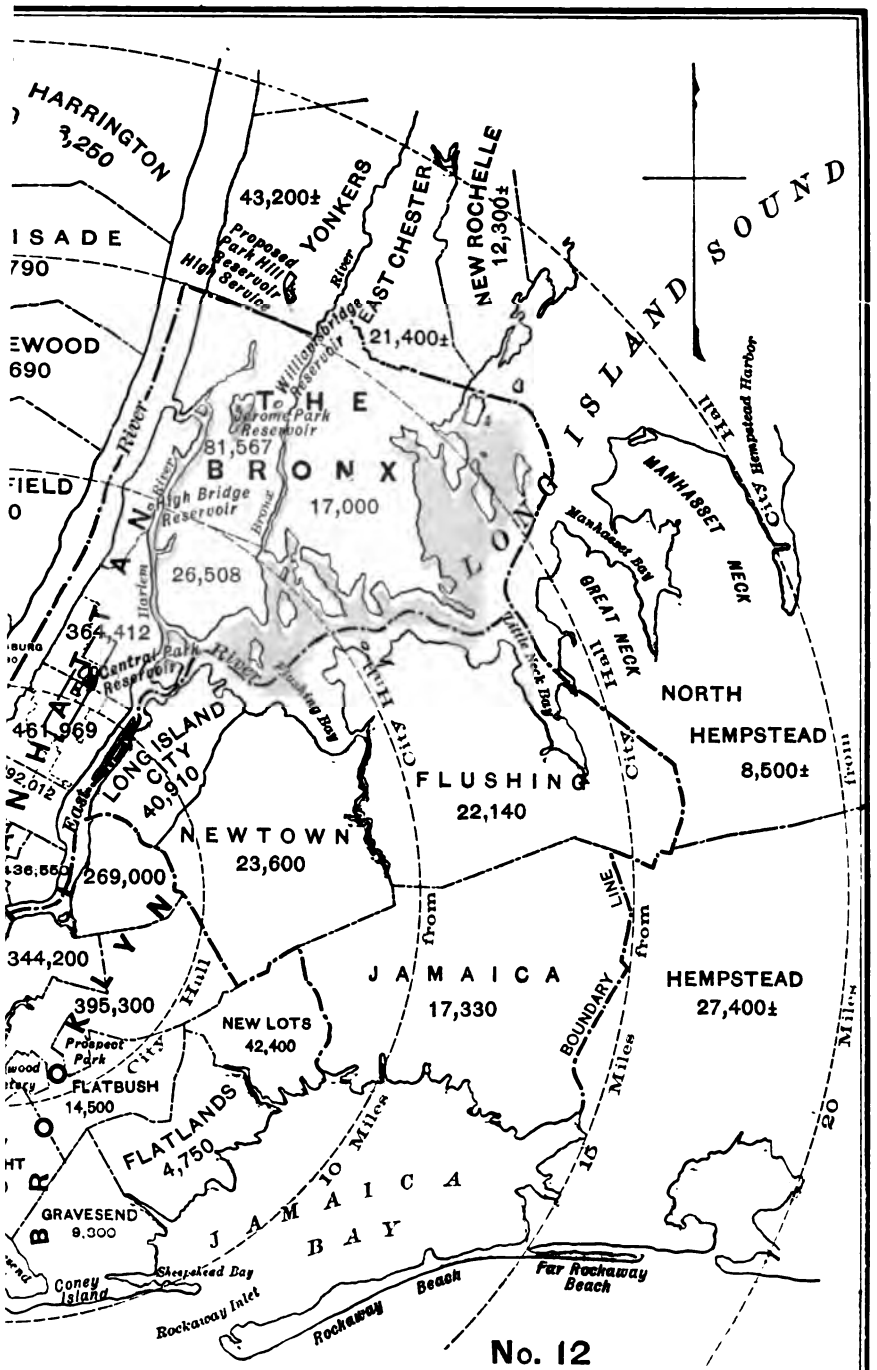
In Cincinnati three other engineers of large experience in water supply, Mr. John W. Hill, Mr. George H. Benzenberg and Mr. Samuel Whinery made a careful investigation for the extension and betterment of the City Water Works in March, 1896, and after carefully considering the conditions in Cincinnati and in other large representative cities, and *assuming that a more earnest effort to prevent unnecessary waste would be adopted by that city*, based their estimate of requirement upon an average consumption of 130 gallons per inhabitant per day.

Considering these matters, and giving particular attention to the fact so clearly brought out by Diagram No. 2, that with the increasing number of fixtures in houses, the legitimate use and also the waste are steadily increasing, I do not believe it safe or prudent to base our estimate of future needs for Greater New York on less than following average consumption per inhabitant per day:

With the greatest effort to restrict waste that public sentiment will probably demand or support.	With Matters Continued as at Present.
In 1905	125 gallons.	140
In 1910	130 "	155
In 1915	135 "	170
In 1920	140 "	180
In 1925	145 "	190
In 1930	150 "	200

It is dangerous to be too hopeful about preventing a large part of the waste, for with those hopes unfulfilled and reservoirs emptied disaster would be beyond remedy.


FRANKLIN
RIDGEWOOD
2,970
WASHINGTON
1,260
MANCHESTER
5,000
SADDLE RIVER
3,660
MIDLAND
2,640
WAYNE
2,100
PEQUANAC
PATERSON
97,350
City Hall
MONTVILLE
City Hall
LITTLE FALLS
2,420
ACQUACKANONCK
3,600
PASSAIC
19,900
NEW BARBADOES
7,280
River
LODI
5,750
City Hall
CALDWELL
2,640
HANOVER
MONTCLAIR
11,750
BLOOMFIELD
8,100
FRANKLIN
8,076
BOILING SPRINGS
1,780
UNION
1,850
NORTH BERGEN
8,430
Hackensack
LIVINGSTON
1,310
WEST ORANGE
5,850
ORANGE
22,800
EAST ORANGE
17,930
BELLEVILLE
7,570
Passaic
KEARNEY
10,490
HARRISON
7,200
CHATHAM
MILBURN
2,760
SOUTH ORANGE
5,110
CLINTON
5,470
NEWARK
215,800
JERSEY CITY
182,710
SUMMIT
4,450
SPRINGFIELD
930
UNION
3,410
CLINTON
Newark Bay
UPPER BAY
WESTFIELD
9,710
CRANFORD
2,150
ELIZABETH
43,835
BAYONNE
19,860
FANWOOD
CLARK
380
LINDEN
2,430
CASTLETON
18,730
NORTHFIELD
11,640
MIDDLETOWN
11,430
RAHWAY
7,950
Rahway River
STATEN ISLAND
WOODBRIDGE
5,800
SOUTHFIELD
7,690
WESTFIELD
R 10,360
PERTH AMBOY
13,030
Raritan River
LOWER BAY
RARITAN BAY



No. 12

DISTRIBUTION OF POPULATION IN THE

METROPOLITAN DISTRICT

ACCORDING TO THE CENSUS OF 1895.

Compiled by J. R. Freeman, C. E., January 1900.

to aid in Estimate of Water Supply required in future by

GREATER NEW YORK

No. 13
PROBABLE INCREASE
OF
POPULATION OF NEW YORK CITY
AND THE
METROPOLITAN DISTRICT

See Appendix No. 4 and Table No. 44,

Prepared by John R Freeman, C. E.
to accompany Report on Water Supply,
January 1900.

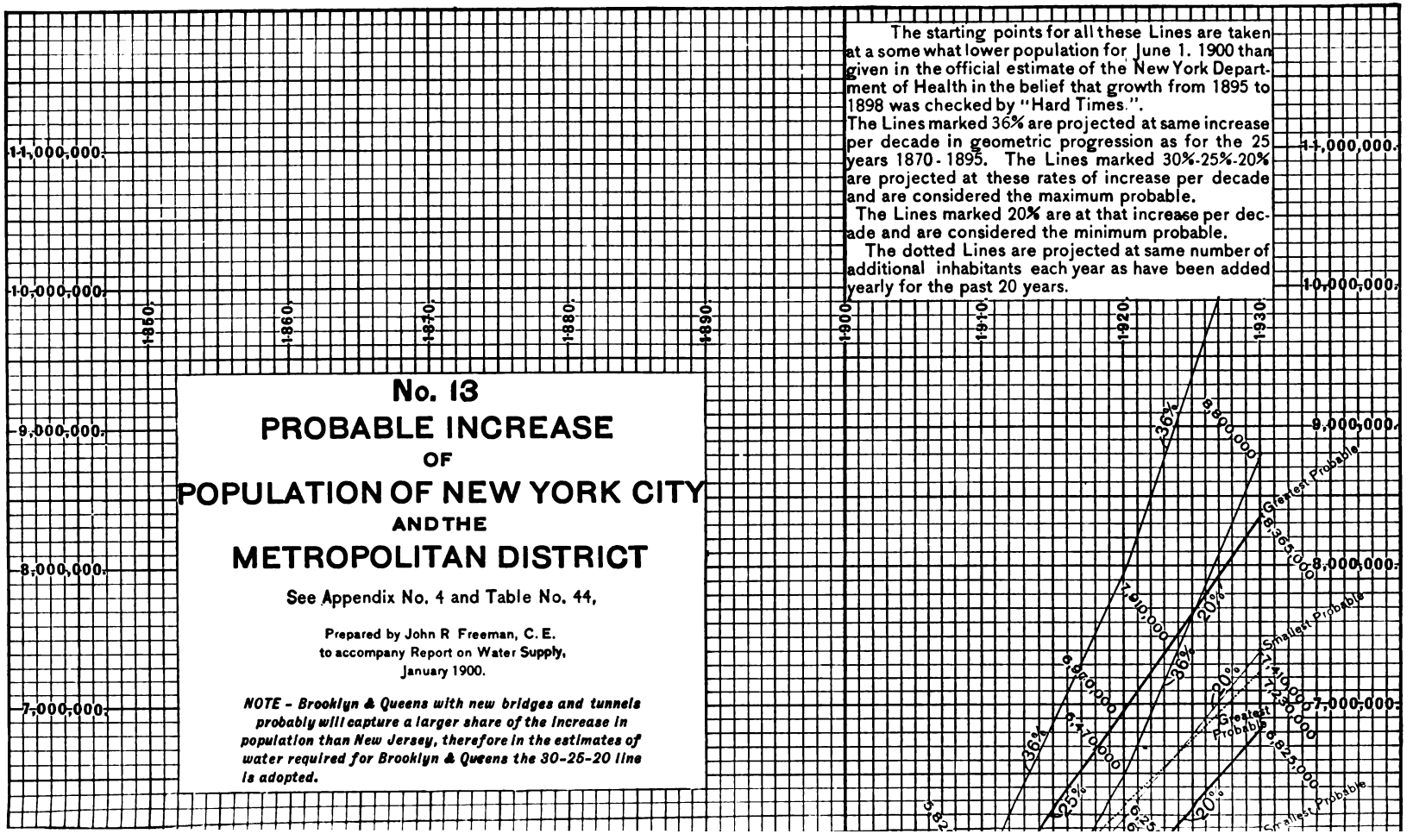
NOTE - Brooklyn & Queens with new bridges and tunnels probably will capture a larger share of the increase in population than New Jersey, therefore in the estimates of water required for Brooklyn & Queens the 30-25-20 line is adopted.

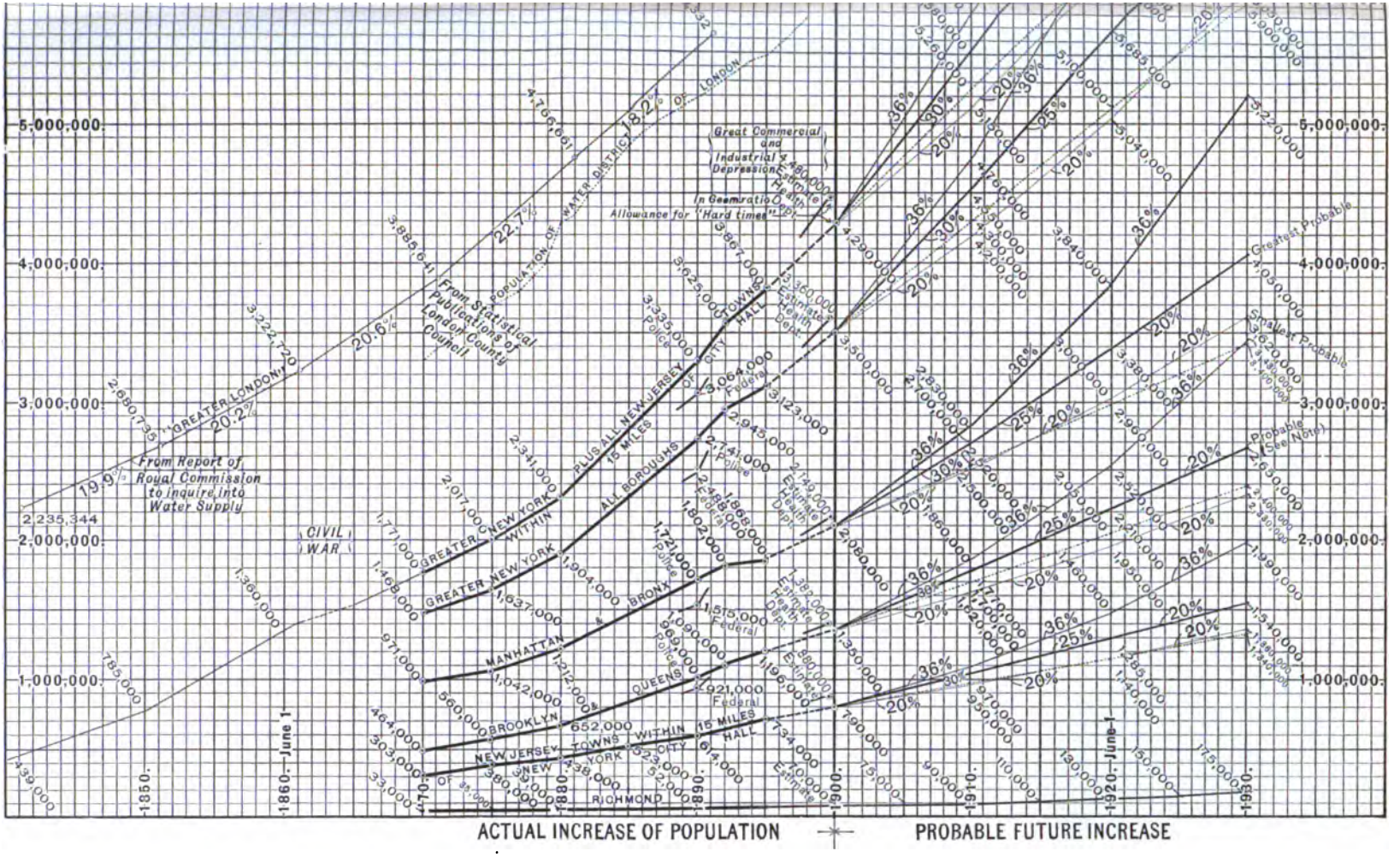
The starting points for all these Lines are taken at a some what lower population for June 1. 1900 than given in the official estimate of the New York Department of Health in the belief that growth from 1895 to 1898 was checked by "Hard Times.".

The Lines marked 36% are projected at same increase per decade in geometric progression as for the 25 years 1870-1895. The Lines marked 30%-25%-20% are projected at these rates of increase per decade and are considered the maximum probable.

The Lines marked 20% are at that increase per decade and are considered the minimum probable.

The dotted Lines are projected at same number of additional inhabitants each year as have been added yearly for the past 20 years.




5,000,000
4,000,000
3,000,000
2,000,000
1,000,000
1850
1860—June 1
1900
1910
1920—June 1
1930
ACTUAL INCREASE OF POPULATION
PROBABLE FUTURE INCREASE
GREATER LONDON
From Report of Royal Commission to inquire into Water Supply
From Statistical Publications of London County Council
POPULATION OF WATER DISTRICT OF LONDON
CITY OF LONDON
CIVIL WAR
Great Commercial and Industrial Depression
Allowance for "Hard times"
TOWNS WITHIN 15 MILES
CITY HALL
Federal
Police
Health Dept.
Estimate
ALL BOROUGHS
PLUS ALL NEW JERSEY
GREATER NEW YORK WITHIN 15 MILES
NEW YORK
BRONX
MANHATTAN
QUEENS
BROOKLYN
NEW JERSEY TOWNS WITHIN 15 MILES
RICHMOND
Greatest Probable
Probable
Smallest Probable
(See Note)

TABLE

FUTURE WATER SUPPLY

In the table below—The **ideal conditions** suppose a meter on every tap, a curb stop-cock on every service pipe, thorough search for leaks as described in report, page 68, by means of a block system of Venturi and Deacon meters; a thorough overhauling and relaying of old and leaky mains; relaying old and leaky service pipes; organized waste inspection and prompt repairs and best of management. The whole being a reasonable, economical, practicable engineering proposition, if public sentiment would demand and maintain it.

YEAR.	1900.			1905.			1910.		
	Ideal conditions.	Best results probably attainable.	Under present conditions.	Ideal conditions.	Best results probably attainable.	Under present conditions.	Ideal conditions.	Best results probably attainable.	Under present conditions.
MANHATTAN AND BRONX.									
Probable population	2,080,000			2,350,000			2,600,000		
Gallons per inhabitant per day	...	125	125	70	125	140	72	130	155
Million gallons required daily	...	265	265	164	293	328	187	338	403
Safe dry year yield of Croton, Bronx and Byram rivers	...	265	265	290	290	290	290	290	290
Addition needed for Manhattan and Bronx	...	...	...	**0**	**3**	**38**	**0**	**48**	**113**
BROOKLYN AND QUEENS.									
Probable population	1,350,000			1,550,000			1,770,000		
Gallons per inhabitant per day (after amply supplied. Assumed at 80% of Manhattan's consumption per inhabitant)	...	90	100	60	100	112	62	104	124
Million gallons required daily	...	122	135	90	150	168	112	187	223
Safe yield, present sources owned by the city, after a series of dry years (per estimate of I. M. de Varona)	...	114	114	114	114	114	114	114	114
Addition needed for Brooklyn and Queens	...	**8**	**21**	**0**	**36**	**54**	**0**	**73**	**109**
Addition possible by extension from Massepequa to Connecticut river, Long Island (per estimate of I. M. de Varona)	...	...	...	100	100	100	100	100	100
Additional supply for Brooklyn that must come from mainland, provided 100 million gallons per day can be taken from Suffolk County	...	...	...	0	0	0	0	0	9
RICHMOND.									
Probable population	70,000			75,000			90,000		
Gallons per inhabitant commonly required by communities under similar conditions, say	...	60	80	50	65	85	52	70	90
Million gallons required daily	...	4	6	4	5	6	5	6	8
Safe capacity of present sources. (This is doubtful after island becomes thickly settled)	...	5	6?	...	5	6?	...	5	6?
Additional supply probably needed. (With all villages properly supplied. Uncertainty about permanence of present supply makes this line of quantities uncertain), say	1	1	.3	1	1	2	1	2	3
Total additional supply needed for all Boroughs	...	**9**	**23**	**1**	**40**	**94**	**1**	**123**	**225**
Amount needed from the north provided 100 million gallons are allowed to be taken from Suffolk County	...	...	...	...	3	38	...	48	125

Quantities of water above are stated in million gallons per 24 hours.

No. 5.

Required for New York City.

The **best results probably attainable** assume meters on half or two-thisds o the taps and a moderate effort at finding and stopping the thousands of leaks in street mains and service pipes, and inspection of house plumbing, say, once a year, and systematic testing of meters.

Under **present conditions** assume domestic services mostly without meters, factories and commercial uses all metered, meters owned by consumer not periodially inspected, curb stop-cocks few, public buildings not metered, conditions of inspection and remedy for leaks averaging just the same as in the average large American city.

1915.			1920.			1925.			1930.		
Ideal conditions.	Best results probably attainable.	Under present conditions.	Ideal conditions.	Best results probably attainable.	Under present conditions.	Ideal conditions.	Best results probably attainable.	Under present conditions.	Ideal conditions.	Best results probably attainable.	Under present conditions.
2,900,000			3,200,000			3,500,000			3,800,000		
74	135	170	76	140	180	78	145	190	80	150	200
214	393	493	243	448	576	273	508	665	304	570	760
290	290	290	290	290	290	290	290	290	290	290	290
0	**103**	**303**	**0**	**158**	**386**	**0**	**218**	**375**	**14**	**280**	**470**
2,000,000			2,200,000			2,430,000			2,600,000		
64	108	136	66	112	144	68	116	152	70	120	160
128	216	272	145	246	317	163	278	365	182	312	416
114	114	114	114	114	114	114	114	114	114	114	114
14	**104**	**158**	**31**	**132**	**203**	**49**	**164**	**251**	**68**	**198**	**302**
100	100	100	100	100	100	100	100	100	100	100	100
0	4	58	0	32	103	0	64	151	0	98	202
110,000			130,000			150,000			175,000		
54	80	100	56	90	100	58	100	120	60	110	130
6	9	11	7	12	14	9	15	18	10	19	23
...	...	...	...	...	...	...	...	...	...	...	...
6?	9	11?	2?	12	14?	4?	15	18?	5?	19	23?
20	**216**	**373**	**33**	**303**	**503**	**53**	**397**	**644**	**87**	**497**	**795**
6	116	272	0	202	403	4	297	544	19	397	695

From the foregoing table it is plain that the providing of a new source of supply that can be so developed ultimately as to safely supply at least 500 million gallons per day should at once be undertaken.

It would be inexcusable to rely on the application of domestic meters as a means of preventing waste while public sentiment is against domestic meters, and inexcusable to invite a water famine five years hence while discussing what ought to be done to educate the public to greater economy in the use of water.

We must be governed by the probabilities of the case, and it is not probable that New York will lead all the large cities in this country much farther than at present in restricting waste, and, therefore, not safe to provide less water per inhabitant than is found to be demanded in all the large American cities, or than eminent engineers have recently recommended for Philadelphia, Cincinnati, Boston and other large cities.

Water procured by any of these large projects is wonderfully cheap to the individual (only a dollar a head a year in New York to-day for all domestic rates), and progress in restricting waste has proved slow.

It is my opinion that the only safe way of avoiding danger and suffering is to provide water on the expectation that the consumption will continue in New York in the future much as heretofore and much the same as in the average large American city.

Therefore, I would respectfully recommend that the column in Table No. 5 headed "under present conditions" be the one used as the basis for the additional supply. Ten or fifteen or twenty years hence I trust that the public in all our large cities will favor complete metering as a prelude to filtration in many of these cities.

The entire interest and maintenance charges for the entire Housatonic project, built in its full magnitude at once and yielding 800 million gallons per day, would cost less than half the annual charge proposed by the Ramapo Company for 200 million gallons per day.

The entire interest and maintenance charges for the entire Housatonic project, built in its full magnitude, would cost only 70 cents per capita per year for the population supplied; and this entire annual cost could be met out of the present surplus revenues of the Water Department.

The following diagram, Fig. 14, gives in graphical form the results of Table No. 5 and shows that if matters go on in the future as in the past, an additional supply of 500 million gallons per day would be exhausted twenty years hence, while if meters can by that time be applied to most domestic

services and a moderate effort be made at finding and stopping the thousands of leaks in street mains and service pipes, this additional supply of 500 million gallons for all boroughs combined, would last thirty years. From this diagram we see further that under the natural growth of the city and with use and waste increasing from year to year at the same rate as heretofore, a supply of about 800 million gallons per day would be exhausted thirty years hence.

We can hope and confidently expect that at some future time public opinion in our large American cities will have become quickened and domestic meters become popular and street waste be strenuously looked after, and that in this way the city of twenty-five or thirty years hence, if it continues to grow at the same rate, can obtain its additional water for use out of a restriction of waste, and **can provide filtration out of the saving in expense for water wasted.**

This diagram indicates that if the prohibition against Brooklyn coming east into Suffolk County remains in force and conditions of use and waste in all the boroughs remain about as at present, a supply of 200 million gallons as proposed by the Ramapo Company would be exhausted within ten years, and the 150 million gallons which the Ten Mile can furnish from a large reservoir with dam at Webatuck, would be exhausted in less than eight years.

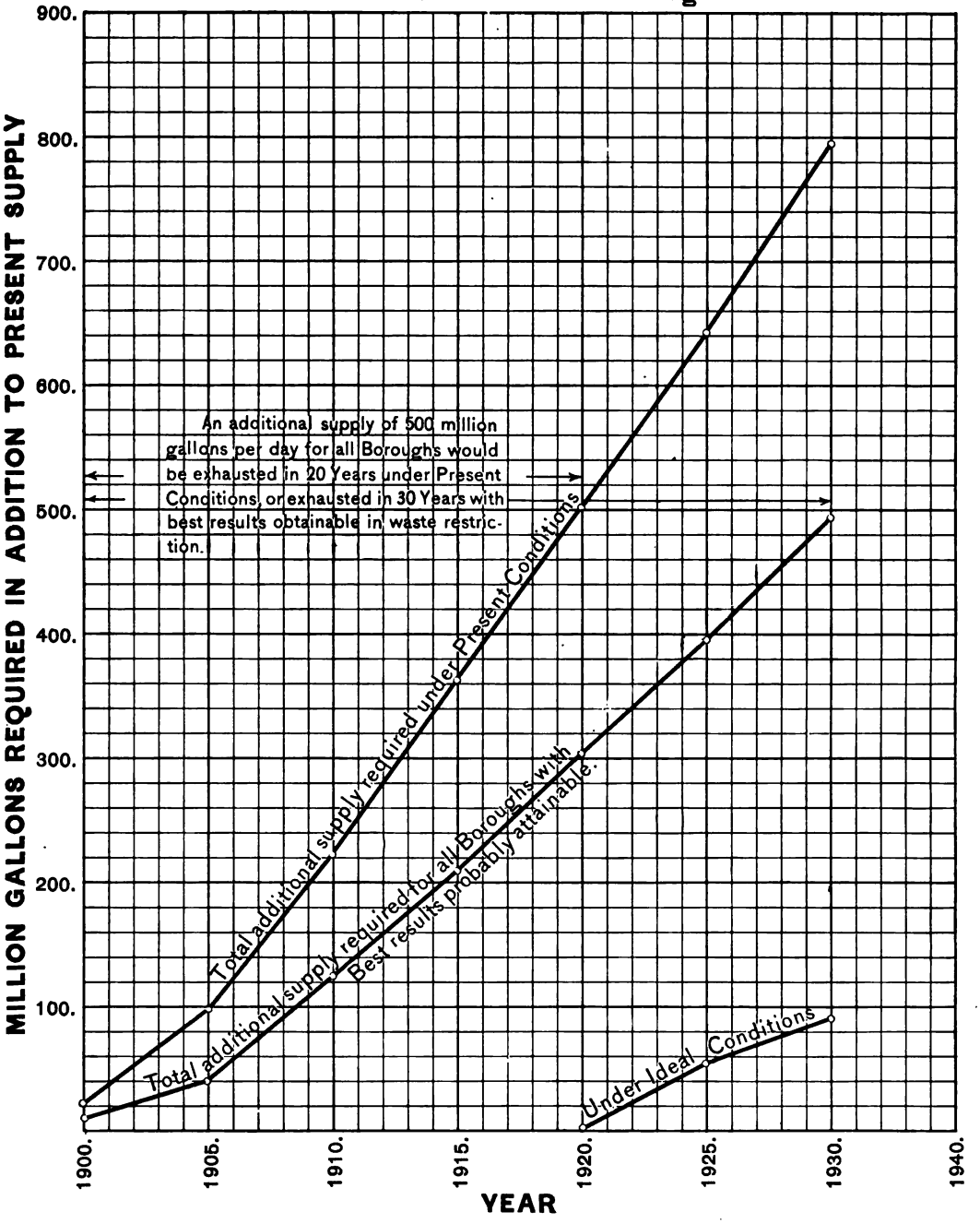
The Ten Mile River and Upper Housatonic combined would probably meet all demands of waste and use, with the natural growth of the city and with present conditions continued for nearly thirty years, and with the education of the public in methods of waste restriction which we may hope will have by that time been attained, this supply will serve abundantly for a period longer than we can now foresee.

No. 14

DIAGRAM OF PROBABLE RATE OF INCREASE

in Demand for additional Water in

Greater New York - All Boroughs



Magnitude of New Supply Recommended.

From a study of the foregoing tables and diagrams I am led to believe that nothing less than a total of 500 million gallons per day should be considered. A supply of 150 to 200 million gallons, if it has to serve for all the boroughs, because of the prohibition against going into Suffolk County, would be exhausted inside of eight or ten years with present conditions continued, or would be exhausted within twelve or fifteen years if a meter was applied to every domestic tap and as much as can be reasonably expected done on replacing leaky service pipes.

I am led to recommend that the investigations take a much broader scope, looking farther into the future than ten or fifteen years, and thus establish a progressive line of development and extension for New York's water supply for a longer future, much as was done for Boston under the direction of the far-sighted Massachusetts State Board of Health, whose plans for ultimate development reach out beyond the Nashua Works now under construction, to the Ware river, the Swift river, and even to the Deerfield, and are so clearly outlined that the development can come step by step and in the end be a harmonious and economical system.

There are all the water sources now on the map that there ever will be and so long as population grows and civilization progresses, the legitimate needs for water must increase, and economy will surely be gained by studying out at the present time the best lines for extension.

THIRD QUESTION.—WATER PRESSURE—FIRE PROTECTION.

The writer, as engineer to an association of large insurance companies, has devoted much time for 14 years past to a study of fire protection, and has had occasion to investigate the progress and the checking of a good many bad fires, and has had occasion to study somewhat broadly the comparative efficiency of the water pressure found in various cities, and has been led to believe that a hydrant pressure of at least 80 pounds per square inch is to be very strongly preferred over a smaller pressure, and that 100 pounds per square inch is probably better, with the modern tendency to higher buildings and with the constantly increasing use of automatic sprinklers in manufacturing and commercial buildings.

It appears feasible to obtain this pressure throughout substantially the whole of New York from a gravity supply; and if the legal authority can be obtained to deliver 800 million gallons per day from the Housatonic, Ten Mile and East Branch sources, with an elevation of 300 feet above the sea at the city limits, this can be delivered at this elevation and pressure at a cost not substantially greater than if delivered at a lower level.

This would do away with all necessity for high service pumping (which now costs the city in Manhattan alone about $70,000 per year for coal and wages, apart from all interest, fixed charges and depreciation); would ultimately save largely in insurance cost (for in the long run rates are always adjusted in proportion to the actual fire loss and to the efficiency of the fire protection); and presents such manifold advantages that I most earnestly recommend that all new work have the utilization of such a pressure in nearly all parts of the city, say, 20 years hence, continually in view, and within 5 years in certain elevated or extra hazardous districts. While I am well convinced that a salt water fire pipe system for the congested commercial district would be an excellent safeguard, and that its cost is remarkably small in proportion to the value of the protection it affords, I venture to suggest that final action on a salt water pipe system be deferred until it is known with certainty what is to be done about securing a gravity supply under a pressure corresponding to an elevation of 300 feet above sea level at the city limits and until the policy with regard to a high pressure distributing reservoir at Park Hill is clearly defined. This need not delay final action on the salt water system longer than until next summer, and meanwhile surveys for it could be in active progress.

As between a salt water fire pipe system like that in Boston and a fresh water high pressure gravity system like that recently constructed in Provi-

dence, I believe the fresh water gravity pressure system is much to be preferred, and I am led to believe that with the present complication of pipes and conduits of various kinds now existing in New York's streets all new piping should be in the direction of simplicity, and that while fire pipes may first be run from the high pressure supply it is best in the long run to furnish the high pressure water to every tap, and as safeguard against breakdown to rely on well-designed cut-off valves and cross connections and on a better type of hydrant. The experience of Syracuse in changing from low pressure to high pressure with no serious difficulty or expense is very instructive.

If a high pressure gravity supply is held by those in authority to be inexpedient, then a salt water pipe system for the limited district in the commercial centre where the concentration of value is greatest is well worth its remarkably small cost. It can be so cheaply and quickly put in that perhaps it is worth its cost for the protection it would afford prior to the completion of the high pressure gravity supply. But I would earnestly recommend that if a salt water pipe system is put in, it should be so designed that it can be turned into a fresh water fire pipe system at any future time.

I am not unmindful of the merits and the low cost of a salt water fire pipe system for use in connection with fire boats, and indeed have myself planned salt water fire pipe systems for some of the largest factories along the Atlantic seaboard, but in my judgment and experience a fresh water gravity system such as it is entirely possible to obtain in New York is much to be preferred.

If the Housatonic is secured there will be more than abundant water to supply all the small uses in street flushing, etc., for which salt water might serve.

The total quantity of fresh water that can be saved by a salt water system is too small a proportion of the entire city supply to cut any material figure in the size of the additional supply of fresh water that is surely needed.

THIRD QUESTION—POSSIBLE NEW SOURCES OF SUPPLY.

A careful study of the maps and a review of the suggestions of many, reveal only the following reasonable possibilities, which are stated in the order of their proximity and economy. These various watersheds are shown on Map No. 1, next the front cover. For close study reference must be had to the larger scale topographical maps of the U.S. Geological Survey.

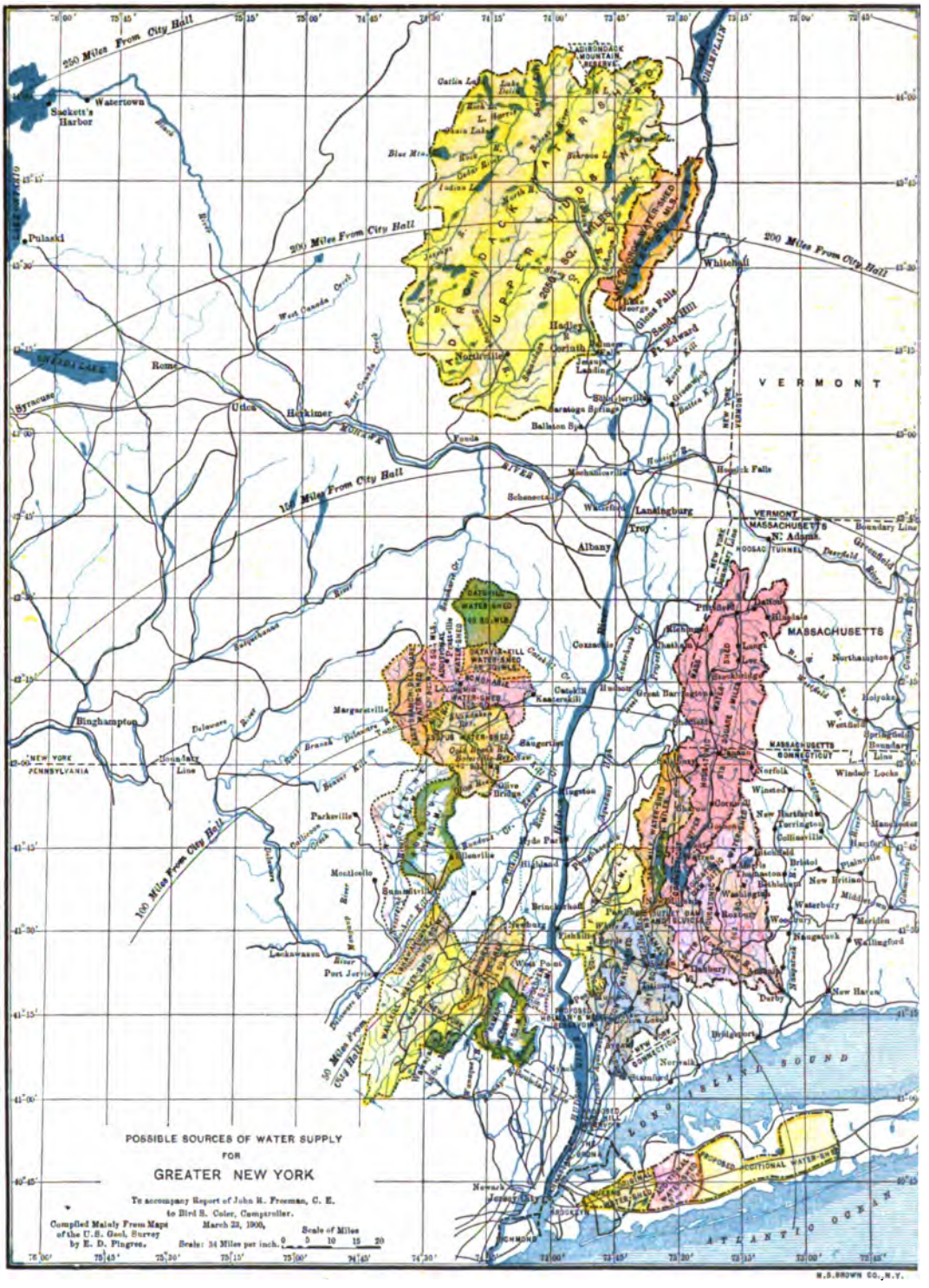
First—THE TEN MILE RIVER.

The Ten Mile river territory lying immediately northwest of the Croton Watershed, with a dam at Webatuck, will cheaply add a safe daily supply of 150 million gallons, equivalent to about ten years' increase at the recent rate of growth for Manhattan and The Bronx only, or equivalent to about seven years' increase, or sufficient until 1907, if Brooklyn and Queens must also share in it. The Ten Mile has the great advantage that merely opening the sluice-gate to be provided at Pawling, it would deliver this water into the Croton Watershed, whence it could run down the stream into Croton Lake and be used in connection with the Croton sources or be conveyed to the city in part through the Old Croton Aqueduct.

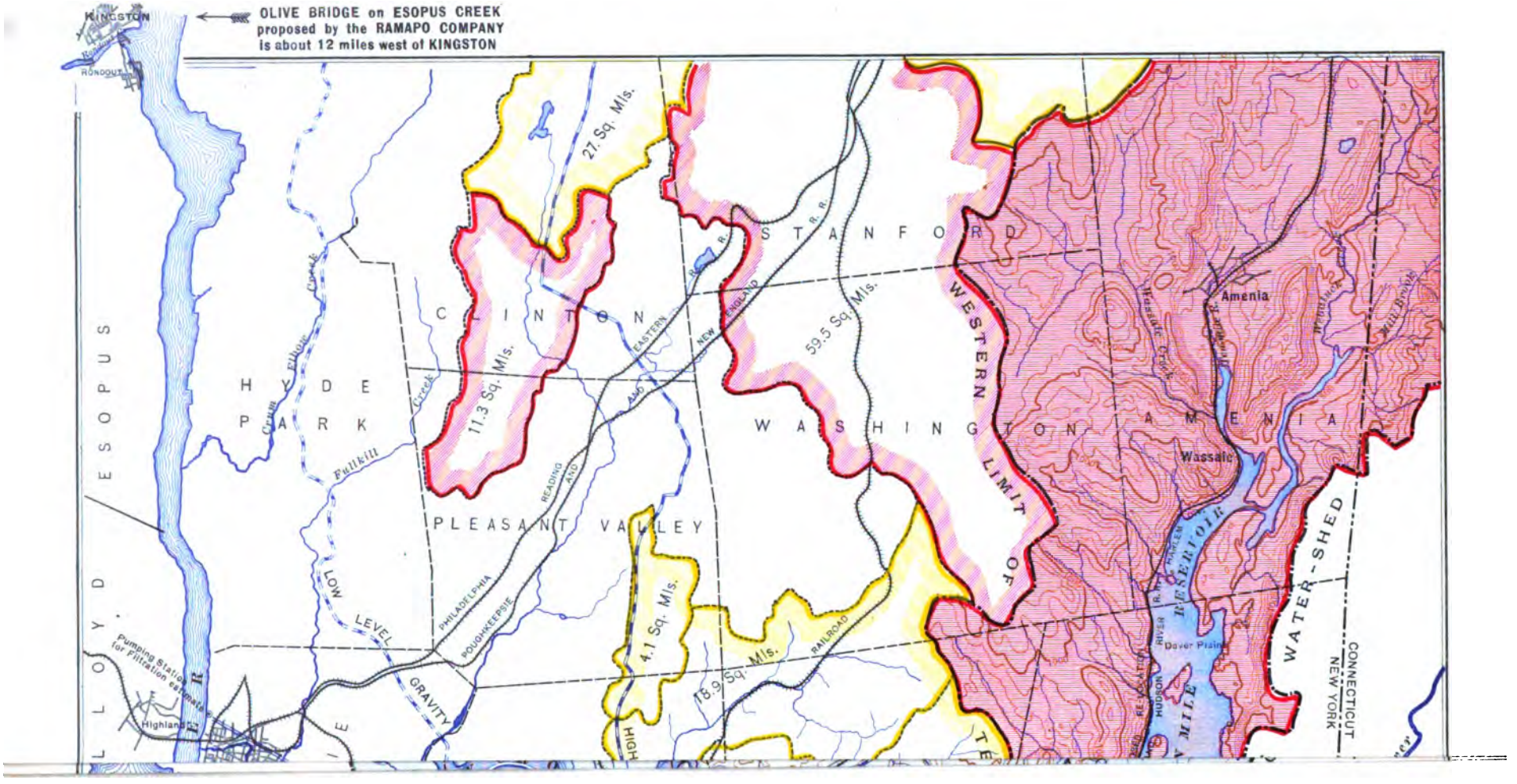
The Old and New Croton Aqueducts have a combined capacity when clean of about 380 million gallons per day, which is 105 millions more than the safe maximum yield of the Croton Watershed; thus leaving a moderate surplus of carrying capacity, subject to some impairment when the aqueducts are not clean, which for a few years could be availed of in carrying a part of the Ten Mile water.

The waters of the Ten Mile river, diverted as proposed, would flow down into the present Croton East Branch Reservoir. This reservoir, fortunately, is at an elevation sufficient, without material sacrifice of storage capacity, to supply an aqueduct line to New York with the 150 million gallons of Ten Mile water, plus the 50 million gallons that the Croton East Branch itself will yield, or 200 million gallons per day, in all, by gravity flow at such a level that it will at the city limits have a head equivalent to an elevation of 300 feet above the sea; thus affording the same elevation tendered by the Ramapo Company, but in my estimates of cost I have provided for a large terminal reservoir near the city limits, for which no equivalent is found in the Ramapo proposition.

The Ten Mile Watershed can be utilized either by itself by a dam at Webatuck, independently of the Housatonic and the Upper Housatonic added later, at that time practically throwing away the cost of the Webatuck



Reduced Copy. See Large Map in Front of Book.


OLIVE BRIDGE on ESOPUS CREEK
proposed by the RAMAPO COMPANY
is about 12 miles west of KINGSTON
KINGSTON
RONDOUT
ESOPUS
HYDE PARK
CLINTON
STANFORD
WASHINGTON
PLEASANT VALLEY
AMENIA
Esopus Creek
Crum Elbow Creek
Fallkill Creek
27. Sq. Mls.
11.3 Sq. Mls.
59.5 Sq. Mls.
4.1 Sq. Mls.
18.9 Sq. Mls.
WESTERN LIMIT OF WATER-SHED
CONNECTICUT NEW YORK
NEW ENGLAND
EASTERN
READING AND
PHILADELPHIA
POUGHKEEPSIE
RAILROAD
LOW LEVEL GRAVITY
Pumping Station for Filtration estimated
Highland
Amenia
Wassaic
Dover Plains
RESERVOIR
HUDSON RIVER
MILE

Dam; which, however, would have been largely, if not wholly, gained in saving of interest. But in view of supplying the additions needed in all boroughs from this source, it may be as well to have these questions of diversion settled at once, instead of in two installments, and so begin on the Housatonic Dam the first of any construction and omit the Webatuck Dam. The cost for the Ten Mile system alone will be found summarized in Table No. 6 and analyzed in detail in Appendix No. 6, page 340.

Second—Upper Housatonic and Ten Mile Rivers.

The Upper Housatonic and Ten Mile rivers combined and diverted by a single dam of substantially the same size as the New Croton Dam, built between granite hills about one mile below Merwinsville, Conn., will deliver 750 or possibly 800 million gallons per day into the Croton Watershed at the remarkably low cost of about ten dollars* per million gallons, cost being reckoned on the entire 750 millions.

This 750 million gallons, or 800 million gallons, when combined with the East Branch of the Croton in Sodom Reservoir, can all be very economically delivered by gravity to New York City under a head equivalent to 300 feet above the sea at the city limits. This head, after allowance for friction in the pipes, is sufficient to afford a water pressure of 100 pounds to the square inch at the street level in front of the New York City Hall (or the water will rise by gravity 230 feet above the street level at the City Hall). This can be done with no expense whatever for pumping, and the cost of nine dollars per million gallons estimated for delivering this entire quantity at the city limits, includes a generous charge for depreciation and maintenance, and also reckons in a sinking fund, which would turn the whole plant over to the city in first class repair and fully paid for at the end of forty years. The total annual cost of all these various items for the whole 800 million gallons would amount to about seventy cents per year for each inhabitant and could be met out of the present surplus of water revenues over expenses for the present water works.

Reckoning the cost from another point of view, *the total annual cost for the whole 750 or 800 million gallons per day, including all interest, repairs, labor, maintenance and sinking fund to pay off the cost, would amount to only one-half the annual rental called for by the proposed Ramapo contract for only 200 million gallons per day.*

* Estimate shows $9.24 per million gallons. If water rights and land damages prove most extravagantly high it can hardly exceed $12.

Or the Housatonic, as compared with the Ramapo proposition, will give about four times the water for one-half the cost, and then at the end of 40 years have the works owned by the city and fully paid for, instead of the city owning nothing at the end of the Ramapo contract.

This magnificent supply possible from the Housatonic would avoid all necessity for house pumps throughout Manhattan; would do away with roof tanks, and afford a most ample supply for fire protection in general and for automatic sprinklers in particular; would avoid all need of a salt-water firepipe system, being very much superior thereto, and would give New York one of the most abundant, cheap and efficient water supplies in the world.

The accuracy of these statements will, I trust, be found satisfactorily demonstrated in Appendix No. 7, page 360 *et seq.*

The estimates of cost are summarized in the following table, No. 6:

NOTE.—For the first 10 or 20 years the waste water that would pass this Housatomic Dam could be very cheaply made to furnish from twenty thousand to ten thousand horse power, available for electrical transmission to any point desired, for lighting or power. The entire mechanical outfit for this purpose would be very cheap in comparison with most power propositions. See Figure 65 next page 370.

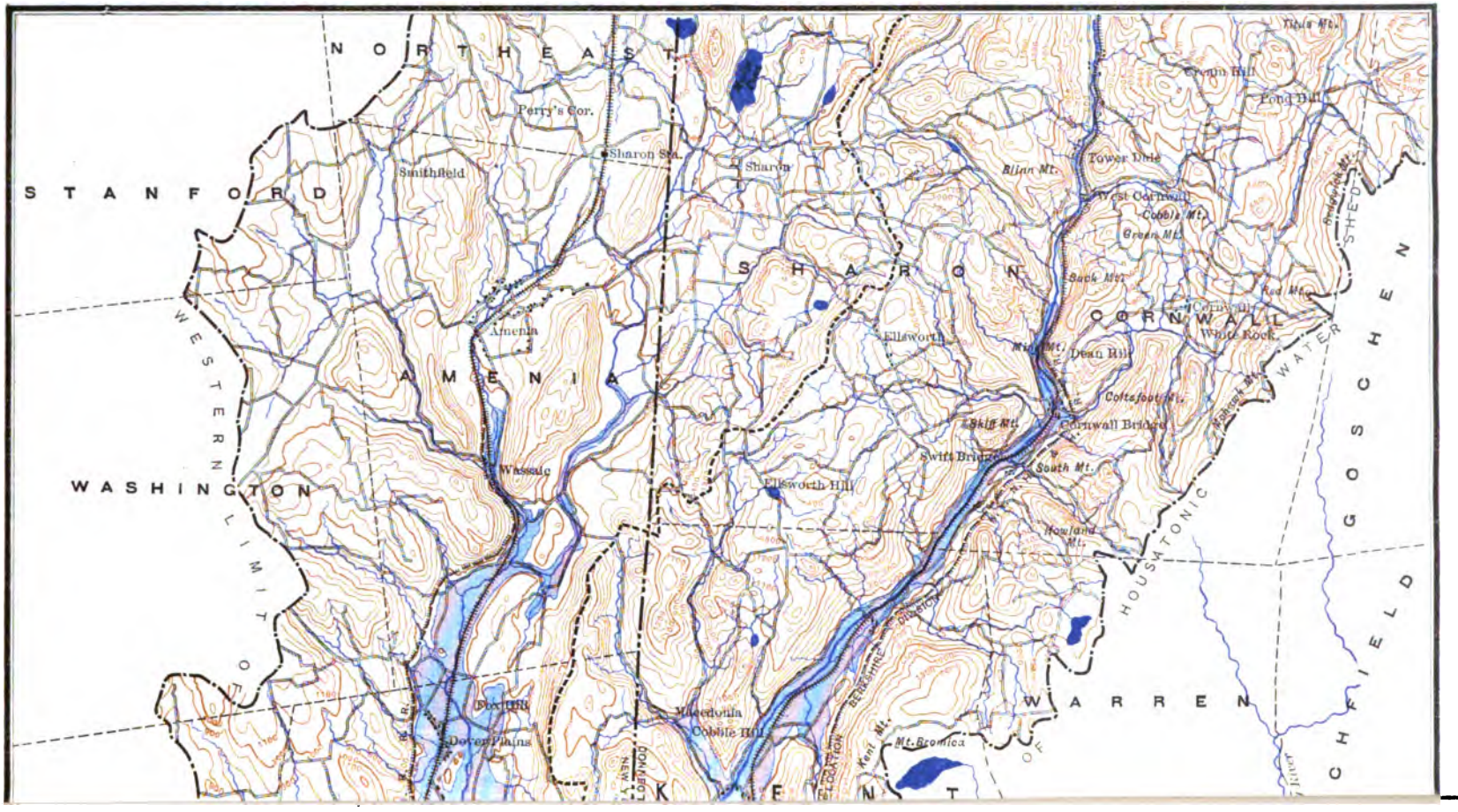

NORTHEAST
STANFORD
WESTERN LIMIT TO
WASHINGTON
AMENIA
SHARON
CORNWALL
WARREN
KENT
Perry's Cor.
Sharon Sta.
Sharon
Smithfield
Amenia
Wassaic
Ellsworth
Ellsworth Hill
Swifts Bridge
Cornwall Bridge
West Cornwall
Dean Hill
White Rock
Tower Dale
Blinn Mt.
Cobble Mt.
Green Mt.
Bald Mt.
Mist Mt.
Skiff Mt.
South Mt.
Cottefoot Mt.
Howland Mt.
Red Mt.
Crown Hill
Pond Hill
Tiffs Mt.
Pinnacle Mt.
Mt. Bromley
Pine Plains
Dover Plains
Wingdale
Millerton
Cobble Hill
Housatonic River
Housatonic Water Shed
Shepaug River
Shepaug Water Shed
Housatonic Water Shed
Ten Mile River
Ten Mile Water Shed
Housatonic Water Shed
Housatonic Water Shed
Housatonic

TABLE NO. 6.

Cost per Year and per Million Gallons of Water, **Ten Mile and Housatonic Rivers,** *Delivered at the City Limits at 300 Feet Elevation Above Sea Level.*

(From reconnaissance only.) With labor and material at prices paid on the new works for Boston.

	Ten Mile.		Ten Mile and Housatonic.			
			With Aqueduct from Sodom to City Limits Only Half Size at First.		With Aqueduct for Whole Quantity.	
Quantity.	Million Gallons.		Million Gallons.		Million Gallons.	
Average quantity of water received from new source	150		400		**750**	
Quantity of Croton water taken from East Branch Reservoir	50		0		**50**	
Average quantity of water delivered at city limits at elevation of 300 feet above sea	200		400		**800**	
Aqueduct capacity provided (with aqueduct clean)	250		500		**1,000**	
Million gallons of water per year from new source	54,750		146,000		**273,750**	
Construction.						
Cost of water rights, structures for gathering, storing and conveying into Croton watershed at East Branch Reservoir	$12,703,000		$26,657,000		**$26,657,000**	
Cost of structures for conveyance and storage into Rye Reservoir (via East line), 22 miles	6,668,000		8,608,000		**11,667,000**	
Cost of structures for conveyance and storage into Park Hill Reservoir (via High Line), 13.8 miles	6,462,000		8,535,000		**10,927,000**	
Total cost of reservoirs and conduits for delivering water to city limits	$25,833,000		$43,800,000		**$49,251,000**	
Operating Expenses.	Per Year.	Per mil. gals.	Per Year.	Per mil. gals.	Per Year.	Per mil. gals
Collection and Storage—						
Interest on entire cost at 3% per year	$381,000	$6 95	$800,000	$5 48	**$800,000**	**$2 93**
Sinking fund to pay off cost in 40 years, at 3%=1.326% per year.	168,000	3 07	353,000	2 42	**353,000**	**1 29**
Taxes and special assessments, 0.4%.	51,000	93	107,000	73	**107,000**	**39**
Operating expenses and repairs on a liberal basis	60,000	1 09	80,000	55	**80,000**	**29**
Extraordinary repairs and depreciation (kept good from maintenance)	.....	.....	.....	.....	.....	.....
Total cost collection and storage.	$660,000	$12 04	$1,340,000	$9 18	**$1,340,000**	**$4 90**
Transportation—						
Interest on entire cost at 3% per year.	$394,000	$7 19	$514,000	$3 52	**$678,000**	**$2 48**
Sinking fund to pay off cost in 40 years at 3%=1.326% per year.	174,000	3 18	227,000	1 55	**300,000**	**1 10**
Taxes and special assessments, 0.2%.	26,000	47	34,000	23	**45,000**	**17**
Operating expenses and repairs on a liberal basis	60,000	1 09	72,000	49	**84,000**	**31**
Extraordinary repairs and depreciation	38,000	69	56,000	38	**76,000**	**28**
Total cost of transportation	$692,000	$12 62	$903,000	$6 17	**$1,183,000**	**$4 34**
Total of annual charges comprising interest, sinking fund, taxes, operation and depreciation	$1,352,000	.....	$2,243,000	.....	**$2,523,000**	.....
Total cost per million gallons for water from Ten Mile and Housatonic rivers delivered at city limits at a head of 300 feet above sea level; estimated on a liberal basis from very complete reconnaissance	.....	$24 66*	.....	$15 35*	.....	**$9 24***
Total, omitting Sinking Fund	1,010,000	18 44	1,663,000	11 40	1,870,000	6 84
	The above is also loaded with transportation for 50 million gallons of East Branch water.		The above includes entire water rights, reservoir cost and full size channel from Pawling to Sodom for the whole 750 million gallons per day.			

*At these prices city could build the works and operate and pay off bonds for entire cost in 40 years and then own the works "free and clear" and in excellent repair.

Third—SUFFOLK COUNTY, LONG ISLAND.

The southern half of Long Island to the east of Massapequa, if the legislative restriction is removed, could probably be made to add from 60 to 120 million gallons per day, by a skillful development of the ground water, equivalent to nearly ten years' natural increase for only Brooklyn and Queens under present conditions, or perhaps sufficient for fifteen years' increase in growth, with all taps metered and with an earnest effort and the best results that can be reasonably expected in waste prevention.

According to Mr. de Varona's estimate, this supply, for pumping, interest and general maintenance, would cost about $39 per million gallons delivered in Ridgewood Reservoir. It is of interest to note that the end of the aqueduct required to bring this water from the Brookhaven Watershed to Brooklyn is as far from Ridgewood Reservoir as the upstream end of the aqueduct from the Housatonic, at Sodom, is from Ridgewood.

There is some doubt thrown on the capabilities of this watershed to yield so much as estimated in the Brooklyn Report, because of the doubt raised by the geological study as to the real gathering ground for the rainfall extending so far north as the divide on the surface topography. This is a matter that can be tested by wash-drill borings and tests of the ground water level and slope.

If the Upper Housatonic can be secured (and it is reasonable to suppose that it can, through the friendly co-operation of the Connecticut Legislature and by fair and liberal payment to those owning adverse rights), then it will not be economical to develop these Suffolk County sources, but will be much cheaper in first cost and in annual expense of maintenance to take water from the proposed terminal reservoir at Park Hill across the East river near Hell Gate, or at some other favorable point. If there is not good prospect of securing the Upper Housatonic, then this territory should be carefully studied and explored by wash-drill borings to locate the level and declivity of the ground water within the yellow gravel, and thus find the true northern limit of the watershed. Possible valleys or erosions in the impervious clay floor underlying the yellow gravel should be sought also by borings.

Fourth—FISHKILL CREEK, ETC.

The small watersheds east of the Hudson river, directly north of the Croton Watershed, can probably be developed to furnish a supply of 150 million gallons per day at the same level as the present Croton supply, and it is probable that by pushing the aqueduct farther north, a larger quantity could be obtained. This supply will require an instrumental survey to give definite information regarding its possibilities, and should certainly be investigated in detail that no time may be lost if negotiations for the Ten Mile and Housatonic should fail.

This source is not large enough for more than a temporary piecing out of the present Croton supply, and does not possess the elevation sufficient for a high level gravity supply. It is relatively expensive for transportation of the water into the Croton Watershed, although the watersheds themselves adjoin; nevertheless it appears to be a practicable source, and the report of a brief reconnaissance upon it will be found in Appendix No. 11, page 466.

An estimate of cost of this source was not made because of lack of time for precise surveys and close study of dam sites.

Fifth—Esopus Creek.

Esopus Creek, draining the southerly slope of the Catskill Mountains, with a watershed of 240 square miles, will, under the storage system as set forth in the filed plans of the Ramapo Company and described in detail by George S. Rice, C. E., in his report to you dated August, 1899, supply safely in a dry year, as I compute it, only about 100 million gallons per day. This water is a mountain water of unusual natural purity and excellence (subject to brief turbidity in the spring and after heavy rains, from the wash of red clay*), but the source is so far from the city that the cost of the long steel pipe line will be large and the expense for reservoirs will be found very heavy.

It may be possible by building the dams higher than indicated on the filed plans and by taking in certain other tributaries at very great expense and with long delay in tunnel work, to increase the safe yield, so that the Esopus and the neighboring Schoharie Creek, would all together yield 200 million gallons per day. The cost of this would be very much greater than for the Ten Mile water, the time necessary to build the works would be longer and the conditions of operation more expensive. I find by personal inspection that with the notable exception of the Olive Reservoir, the proposed reservoirs along Esopus Creek have dam sites that are not promising in visible indications of a good safe foundation for such large dams as would be required to develop the yield to more than 100 million gallons per day. It is, of course, possible that expensive test-pits might find the ledge within practicable reach. The head or pressure given by the Esopus water at the New York City limits would be no greater than that of the Ten Mile river water under the plans hereinafter proposed.

In Appendix No. 9 will be found the report of a brief reconnaissance in the Esopus Watershed, together with rough estimates of cost of the necessary storage and aqueducts.

A summary of these estimates is given on the following page in Table No. 7, from which it appears that the city could itself probably develop the water supply from this territory, pay all interest, all maintenance charges, make all repairs, and also pay each year a contribution to a sinking fund which would pay off the entire cost in 40 years and leave the whole then in good repair and owned by the city free and clear at a cost of, per million gallons, less than half that which the City came near having to pay under the Ramapo contract, or probably for about $25, instead of $70, while if sinking fund for paying off cost is omitted, thus putting it strictly on the basis of the Ramapo proposition, the cost to the City per million gallons would be probably not less than $17.12, and not more than $21.24 per million gallons instead of $70.

* Although under present conditions of rapid flow the stream quickly runs itself clear there is much doubt if large storage reservoirs would not remain permanently discolored. Although the days of flood and red wash are few, the volume of water that flows in these few days may amount to nearly half the flow of the entire year. I noted various small discolored stagnant pools that had apparently failed to clear in long standing. This matter should be investigated by collecting samples in connection with gaugings of flow.

TABLE NO. 7.

Cost per Million Gallons of Water from **Esopus Creek** *and Adjacent Watersheds, Delivered at New York City Limits at* 300 *Feet Elevation Above Sea.*

NOTE.—This estimate is made between high and low limits because of incomplete information as to certain features of the work.	To deliver a daily average of *200 million gallons* of water at city limits at elevation of 300 feet through conduits of 250 million gallons capacity when clean. Tunnels and masonry aqueducts to be full size from the start, siphons and pipes deferred until needed.			
Quantity.	200 million gallons per day, 73,000 million gallons per year.			
Construction.	Not likely to cost less than the value given below if structures are first-class in design, material and workmanship.		Not likely to cost more than the value given below, with the very best of design, material and workmanship.	
Cost of structures for storing and conveying water on watersheds of Esopus, Schoharie and Batavia Creeks, viz.:				
30,000 million gallons storage	at $240	$7,200,000	at $425	$12,750,000
2-7 foot tunnels, 11 miles		1,460,000	+ 50%	2,190,000
Cost of structures for conveyance, Olive Bridge Reservoir to city limits	Part steel pipe, " tunnel, " C.& C. aqueduct	22,000,000	.....	26,000,000
Total cost of reservoirs and conduits for delivering water at city limits		$30,660,000		$40,940,000
Operating Expenses.	Total cost per year.	Per mil. gals.	Total cost per year.	Per mil. gals.
Collection and Storage—				
Interest on entire cost of storage at 3% per year	$260,000	$3 56	$448,000	$6 14
Sinking fund to pay off cost in 40 years at 3% = 1.326% per year	115,000	1 58	198,000	2 71
Taxes and special assessments 0.4%	35,000	48	60,000	82
Operating expenses and repairs on a liberal basis * of city management	60,000	82	60,000	82
Extraordinary repairs and depreciation (kept good from maintenance)	.....	00	.....	00
Total cost collection and storage	$470,000	$6 44	$766,000	$10 49
Transportation—				
Interest on entire cost of conduit at 3% per year	$660,000	$9 04	$780,000	$10 68
Sinking fund to pay off cost in 40 years at 3% = 1.326% per year	292,000	4 00	345,000	4 73
Taxes and special assessments 0.2%	44,000	60	52,000	71
Operating expenses and repairs on a liberal basis * of city management	86,000	1 18	124,000	1 70
Extraordinary repairs and depreciation	105,000	1 44	124,000	1 70
Total cost of transportation	$1,187,000	$16 26	$1,425,000	$19 52
Total of annual charges, comprising interest, sinking fund, taxes, operation and depreciation	$1,657,000*	.....	$2,191,000*	.....
Total cost per million gallons for water from Esopus Creek district delivered at city limits at a head of 300 feet above sea level	.....	$22 70*	.....	$30 01*
Total exclusive of Sinking Fund	$1,240,000	17 12	$1,648,000	22 67

*At these prices city could build the works and operate and pay off bonds for entire cost in 40 years, and then own the works "free and clear" and in excellent repair.

* A private company, giving less attention to the neat appearance of the property, to policing and elaborate sanitary inspections, could probably reduce this cost of operation one-half.

At the proposed Ramapo Company's contract price of $70 per million gallons this water would have cost the city 73,000 x $70 — $5,110,000 per year.

While the actual cost of production at the City's low rate of interest, by estimate above, exclusive of sinking fund, would average only $1,444,000 per year.

Sixth—FILTERED WATER FROM THE HUDSON AT POUGHKEEPSIE.

An almost unlimited quantity can be taken just above Poughkeepsie and purified by filtration, just as Poughkeepsie has done for its own public water supply for upwards of twenty years, or just as Philadelphia is about to purify the Schuylkill and Delaware water for general use.

A reconnaissance and estimate of the cost of this plant was made and is given in Appendix No. 10, from which it will be seen that 200 million gallons per day of filtered water from the Hudson river delivered at 300 feet elevation could be had for $40 to $50 per million gallons, and for about $7 per million gallons less than this sum if delivered at the lower level of the present Croton supply, thereby saving expense of pumping.

This will be much more expensive than water from the Upper Housatonic. A supply of 200 million gallons per day from the Hudson at Poughkeepsie costing about as much as 800 million gallons per day from the Ten Mile and Housatonic.

The cost per million gallons and the annual expenditure in interest, etc., will be more for this filtered Hudson water than for water from the Adirondacks, but the total sum required to be expended for water rights and original construction, and therefore, the first installment of the total bond issue will be very much smaller.

The ultimate total expenditure for pumps and filters and all accessories of 800 millions daily capacity makes an enormous sum, rising almost into the class with the cost of the Adirondack project.

I made estimates of this, but as the question of date of construction for the future pumps and filters which form so large a proportion of the total cost was rather indefinite, and the question of present value for these deferred expenditures somewhat obscure. I have not reproduced these figures in the report.

A rough estimate similar to the opposite table, but for a daily supply of 400 million gallons. Without deduction for present value of deferred pumps, etc., will be found on page 465A.

TABLE NO. 8.

Probable Cost per Year and per Million Gallons **Filtered Water from the Hudson River at Poughkeepsie,** *Delivered in New York at City Limits.*

(From partial reconnaissance only.)

For plant of 250 gallons, nominal capacity, delivering 200 million gallons per twenty-four hours.

Quantity.	For 300 feet head at city limits.		For 135 feet head at city limits.*	
Million gallons per year delivered at city limits, 200 x 365=	73,000 million gallons.		73,000 million gallons.	
Construction.				
Cost of pumping plant, including intake works, docks, coal pockets, lands, etc., complete	$4,830,000		$3,824,000	
Cost of filtration works, lands, etc., complete	6,800,000		6,800,000	
Cost of structures for conveyance and storage, Poughkeepsie to city limits.	18,733,000		19,802,900	
Park Hill High Pressure Distributing Reservoir	3,696,000		0	
Contribution toward cost of Adirondack Reservoirs, for preventing saltness in drought	1,000,000		1,000,000	
Total cost of works for pumping, filtration and conveying water to the city limits on smallest basis reasonable in absence of surveys	$35,059,000		$31,426,900	
Operating Expenses.	Total cost per year.	Per mil. gals.	Total cost per year.	Per mil. gals.
Collection and Pumping—				
Interest on entire cost of structures and land at 3% per year	$174,900	$2 38	$144,700	$1 97
Sinking fund to pay off cost in forty years at 3%= 1.326% per year	77,000	1 05	64,000	88
Taxes and special assessments at 0.4% of entire cost	23,300	32	19,300	26
Extraordinary repairs and depreciation at 3% of entire cost	174,900	2 38	144,700	1 97
Coal, labor and supplies, per million gallons raised 1 foot at 3.5 cents	1,150,000	15 75	715,400	9 80
Total cost of pumping	$1,600,000	$21 88	$1,088,000	$14 88
Filtering—				
Interest on entire cost of stuctures and land at 3% per year	$204,000	$2 80	$204,000	$2 80
Sinking fund to pay off cost in forty years at 3%= 1.326% per year	90,200	1 24	90,200	1 24
Taxes and special assessments at 0.4% of entire cost	27,200	37	27,200	37
Extraordinary repairs and depreciation	68,000	93	68,000	93
Maintenance, operation, labor and supplies at $3 per million gallons	219,000	3 00	219,000	3 00
Total cost of filtering	$608,400	$8 34	$608,400	$8 34
Transportation (Conduits and Terminal Reservoirs)—				
Interest on entire cost of structures and land at 3% per year	$672,900	$9 22	$594,000	$8 14
Sinking fund to pay off cost in forty years at 3%= 1.326% per year	297,400	4 07	262,600	3 59
Taxes and special assessments at 0.2% of entire cost	44,900	61	39,600	54
Extraordinary repairs and depreciation	107,600	1 47	72,000	99
Operating expenses and repairs on a liberal basis	110,000	1 51	103,000	1 41
Total cost of transportation, interest and maintenance, etc., Adirondack Reservoir	$1,232,800	$16 88	$1,071,200	$14 67
Total of annual charges comprising interest, sinking fund, taxes, operation and depreciation	$3,441,200	.....	$2,767,600	.....
Total cost per million gallons of water, from Hudson river at Poughkeepsie, filtered and delivered at New York City limits, including Park Hill Reservoir	.....	$47 10	.....	$37 90
Total exclusive of Sinking Fund	2,976,400	40 77	2,350,800	32 20

*At these prices, city could build works and operate and pay off bonds for entire cost in 40 years, and then own the works "free and clear" and in excellent repair.

*Jerome Park Reservoir is 1.7 miles south of point of delivery at city limits, with high water elevation 131.5 feet above sea.

Seventh—THE UPPER HUDSON, OR ADIRONDACKS.

The Upper Hudson, or Adirondacks, at a total cost which is prohibitive, in view of the cheaper Housatonic, but which might not be insurmountable if there were no nearer sources, would afford from 200 to 1,000 million gallons per day of excellent water under an elevation of 300 feet above the sea at the city limits, and with no expense for pumping. An aqueduct line for bringing the Adirondack water from Corinth to the city limits appears entirely feasible.

Although the first cost of an Adirondack supply is extremely large, it is of interest to examine the estimate of cost of this water delivered, as an illustration of the effect of the present low rates of interest upon the cost at which water on a large scale can be developed.

The results of my reconnaissance and a full statement of the basis of my estimates, showing the margin of uncertainty that exists, will be found in Appendix No. 12. A brief summary showing the cost per million gallons is given in Table No. 9, on the following page.

TABLE NO. 9.

Cost per Year and per Million Gallons of Water, **Adirondack Project,** *Palmer's Falls to New York City Limits, Delivered at Elevation of* 300 *Feet Above Sea.*

(From reconnaissance only, without surveys in detail.)

	Million Gallons.		Million Gallons.		Million Gallons.	
Average quantity of water delivered at city limits at elevation of 300 feet above sea	200		400		800	
Aqueduct capacity provided (with aqueduct clean)	250		500		1,000	
Million gallons of water per year from this source	73,000		146,000		292,000	
Construction.						
Assumed cost of water rights, structures for gathering and storing the waters of the upper Hudson river and tributaries (probably low rather than high)	$4,000,000		$8,500,000		$15,000,000	
Cost of structures for conveyance and storage, Palmer's Falls to city limits	55,591,900		81,056,900		113,566,000	
Terminal reservoir, at Park Hill	3,696,000		3,696,000		5,542,000	
Total cost of reservoirs and conduits for delivering water at city limits, on smallest basis reasonable in absence of surveys	$63,287,900		$93,253,000		$134,108,000	
Operating Expenses.		Per mil. gals.		Per mil. gals.		Per mil. gals.
Collection and storage—						
Interest on entire cost at 3% per year	$120,000	$1 64	$255,000	$1 75	$450,000	$1 54
Sinking fund to pay off cost in 40 years, at 3% = 1.326% per year	53,000	73	113,000	77	199,000	67
Taxes and special assessments, 0.4%	16,000	22	34,000	23	60,000	20
Operating expenses and repairs on a liberal basis	80,000	1 10	80,000	55	80,000	27
Extraordinary repairs and depreciation (kept good from maintenance)	.....	0 00	.....	0 00	.....	0 00
Total cost collection and storage	$269,000	$3 69	$482,000	$3 30	$789,000	$2 68
Transportation—						
Interest on entire cost at 3% per year	$1,779,000	$24 37	$2,542,000	$17 41	$3,573,000	$12 25
Sinking fund to pay off cost in 40 years, at 3%=1.326% per year	786,000	10 77	1,124,000	7 70	1,579,000	5 41
Taxes and special assessments, 0.2%	118,000	1 62	169,000	1 16	238,000	81
Operating expenses and repairs on a liberal basis	252,000	3 45	316,000	2 16	396,000	1 36
Extraordinary repairs and depreciation	222,000	3 04	347,000	2 38	502,000	1 72
Total cost of transportation	$3,157,000	$43 25	$4,498,000	$30 81	$6,288,000	$21 55
Total of annual charges, comprising interest, sinking fund, taxes, operation and depreciation	$3,426,000*	.....	$4,980,000*	.....	$7,077,000*	.....
Total cost per million gallons for water from Adirondack and Upper Hudson river, delivered at city limits at a head of 300 feet above sea level. Possibly detailed surveys would increase these costs 10 or 15%	.....	$46 94*	.....	$34 11*	.....	$24 23*
Total cost exclusive of Sinking Fund	2,587,000	35 44	3,747,000	25 64	5,299,000	18 15

* These are the smallest probable costs at which city could build works and operate and pay off bonds for entire cost in 40 years, and then own the works "free and clear," and in excellent repair.

Eighth—WALLKILL AND ADJOINING WATERSHEDS.

The Popolopen, Moodna, Wallkill and Shawangunk offer an attractive line of survey for a supply west of the Hudson, since these streams, below their point of diversion lie wholly within the State of New York and the diversion of their waters is thus free from interstate complications. The water from each of these streams could be delivered by gravity flow at the 300 foot elevation proposed by the Ramapo Company, and also proposed in the plans that I have submitted for the Ten Mile and Housatonic. The first impression from the map is that the same aqueduct could ultimately be projected farther northwest, to take in the headwaters of the Rondout and Delaware, and perhaps even the Esopus, but the field examination is much less promising, because of questionable quality of the Wallkill and Moodna water; the probable loss by percolation at the Wallkill dam site and the small area of the Shawangunk Watershed at suitable elevation, and the Rondout and the Delaware branches are found very difficult of access because of the high land intervening.

It appears that Popolopen could be developed to yield a safe supply of excellent water amounting to 20 million gallons per day, and that the Big and Little Modna could be made to add about 100 million gallons. The lay of the land is such that the Popolopen and Moodna would naturally be led into the same aqueduct.

The Wallkill, by availing of the vast reservoir of 40 or 50 square miles that would be had by re-flooding the "Drowned Lands," would under ordinary rules for yield of watersheds of a given area and given storage capacity yield of 315 to 365 million gallons per day, or just about half the capacity of the Ten Mile and Housatonic, if there was no percolation to lower levels, but at a cost somewhat greater than the entire cost of the 800 millions by the Housatonic project. The Shawangunk developed in connection with the Wallkill would add 35 millions, bringing the total for Wallkill and Shawangunk to 400.

In brief, as to *quantity*, if the possibility of percolation at the dam site could be disregarded, it would appear feasible to obtain the following safe yields in the cycle of low rainfall, by liberal expenditure for storage and carrying the development to the very highest reasonable limit:

From	Popolopen Watershed.....	20	million	gallons	per	day.
"	Moodna (Big and Little).....	100	"	"	"	"
"	Wallkill	365	"	"	"	"
"	Shawangunk	35	"	"	"	"
	Total	520	"	"	"	"

I regret to have to report that such personal examination as I have been able to make of this in the field together with the reports presented by my assistants, fail to sustain in the promise held out by the map, particularly in regard to both the amount of the water obtainable and in the probable quality of the water from the Wallkill and the Moodna.

The reports of the Assistant State Geologist, Dr. Heinrich Ries (see Geological Survey of N. Y., Orange Co., 1895, p. 474; also pp. 465 and 466), speak of the depth of the rich, black soil of the old swamp bottom of the Drowned Lands in a way that is far from hopeful to the seeker after potable water. Half of this proposed reservoir area was the bottom of a swamp drained only about 50 years ago, and now forming as fertile ground as can be found in the State. Its deep black soil of swamp muck is said to be from 5 to 50 feet deep over very large areas, hopelessly extensive for either stripping or for covering with gravel.

I found it easy to push a pole down 7 feet or more at almost any point tried upon the large areas devoted to onion culture, and by digging small test pits at widely scattered points found the soil to be mainly decomposed peat and with a very large proportion of the peat fibres uniformly present at a depth of 3 feet. Samples were collected for test, and report of analyses showed nearly 90% of organic matter. Several of the farmers told me there was "no bottom" to this muck. That "a pole could be pushed down 25 feet," etc. As far up as Owen's Station, in New Jersey, I was able to easily push the longest pole available down 10 feet.

I was for some years Water Commissioner in a town that for one of its reservoirs, had flooded a swamp, and had the possible danger of developing such sources so forcibly brought to my attention that I would advise proceeding with much caution before recommending the flooding of the Drowned Lands as a storage basin for New York. The engineer who stands foremost in this country in the development of the science and art of filtration, advises me that it is by no means yet proved that filtration is a practical and sufficient remedy for rendering waters subject to rank growths of algæ, potable and attractive; and the Massachusetts State Board of Health, without doubt the foremost authority of its kind in America on these matters, is understood to have recently advised the City of Springfield, Mass., to seek a new source rather than build filters for remedying the bad tastes that develop frequently in the water of its Ludlow Reservoir because of a swampy bottom, and which in nearly 30 years of use has not improved.

It is possible that one or two years' careful expert study, for example, like the two years of investigation given to the filtration problem in Louisville and the extended study given to filtration in Cincinnati, which cost in the one case, all told, nearly $100,000, and in the other case $30,000, would prove that it was safe to go ahead with the development of the Wallkill by flooding the Drowned Lands for storage and purifying the water by filtration, but I regard the probabilities as strongly against good water from this source.

A summary of the estimates of cost will be found in the following table, No. 10.

This is based on a hurried but careful reconnaissance under my supervision by Horace Ropes, C. E., and rests on scant data. That there may be no mistake about the method of estimate and the margin of uncertainty, I present abstracts from his report in Appendix No. 14. I have personally gone over much of this ground, but the estimates and the main reconnaissance in this region were made by Mr. Ropes.

TABLE NO. 10.

Cost per Million Gallons of Water from **Wallkill and Shawangunk.**

Taking the most favorable, reasonable view on all doubtful questions.

Supply of 400 mil. gals. daily — 146,000 mil. gals. per year. Delivered at New York City limits at 300 feet elevation above sea. Estimates from brief reconnaissance.

Construction (very rough estimate).

Cost of Wallkill Reservoir	$12,350,000
Cost of Shawangunk works ($1,873,000) deferred 18 years	1,000,000
Cost of filtration works, built at first, 100 mil. gal.	2,350,000
Cost of filtration works, deferred 8 years, 100 mil. gal., present value	1,780,000
Cost of filtration works, deferred 15 years, 156 mil. gal., present value	2,250,000
Cost of conduits, part deferred	24,868,000
Cost of Park Hill Reservoir (north basin only)	3,696,000
Total cost of works for storage, filtration and conveyance for 406 mil. gal. supply	$48,294,000

Rough Estimate from Brief Reconnaissance.	TOTAL COST PER YEAR.	COST PER MIL. GALS.
Operating Expenses.		
Collection and Storage—		
Interest on entire cost of structures and lands at 3% per year	$400,000	$2 74
Sinking fund to pay off cost in 40 years at 3%—1.326% per year	177,000	1 21
Taxes and special assessments at 0.4% of entire cost	54,000	37
Operating expenses and repairs on a liberal basis	60,000	41
Extraordinary repairs and depreciation (kept good from maintenance)	.....	.....
Total cost for collection and storage	$691,000	$4 73
Filtration (of 285 mil. gal. daily — 104,000 yearly)—		
Interest on entire cost of structures and lands at 3% per year	$191,000	$1 31
Sinking fund to pay off cost in 40 years at 3%—1.326% per year	84,000	0 57
Taxes and special assessments at 0.4% of entire cost	25,000	0 16
Extraordinary repairs and depreciation	100,000	0 68
Maintenance, operation, labor and supplies at $3 per mil. gals.	312,000	2 14
Total cost of filtering	$712,000	$4 86
Transportation (including Park Hill Reservoir)—		
Interest on entire cost of structures and lands at 3% per year	$857,000	$5 89
Sinking fund to pay off cost in 40 years at 3%—1.326% per year	379,000	2 60
Taxes and special assessments at 0.2% of entire cost	$57,000	$0 39
Extraordinary repairs and depreciation	134,000	92
Operating expenses and repairs on a liberal basis	100,000	68
Total cost of transportation	$1,527,000	$10 48
Total of annual charges comprising interest, sinking fund, taxes, operation and depreciation (on lowest reasonable basis)	$2,930,000	.....
Total cost per mil. gals. of water from the Wallkill and Shawangunk rivers delivered at city limits at head of 300 feet above sea (on lowest reasonable basis and supposing full quantity obtainable)	.....	$20 07

At these prices City could build the works and operate them and pay off bonds for entire cost in 40 years, and then own the works "free and clear" and in excellent repair.

The quantity and the quality of this water are at present doubtful.

Ninth—RAMAPO RIVER.

The Ramapo river, combined with the Mahwah, at a cost probably not far from $15,000,000, could be made to deliver about 65 million gallons at the city limits at a head of nearly or quite 300 feet above the sea. The Ramapo river flows south into New Jersey and is a tributary of the Passaic river, on which valuable rights are understood to be in the control of interests allied to the East Jersey Water Company, and its diversion would raise all the Inter-State questions that have been discussed in connection with the diversion of the Ten Mile river and with less prospect of favorable solution. I have reviewed the possibilities of this source to some extent in Appendix No. 13.

Tenth—DELAWARE RIVER.

The Delaware river at Port Jervis is less than sixty miles distant from New York in an air line and is at Port Jervis at an elevation of about 415 feet above sea level, and is ample in elevation for a supply delivered by gravity flow to elevation 300. While the main river is not very promising, certain of its tributaries, the Beaver Kill, Neversink, Bashers Kill, the East and West Branch of the Delaware, are among the possibilities, but at a cost which, according to a reconnaissance made by one of my assistants, puts it beyond the limits of reasonable projects. The diversion of these streams which flow into other States would raise the Inter-State questions previously referred to.

In all these explorations west of the Hudson we were embarrassed by lack of time, and particularly by the absence of good topographical maps, much more than in our reconnaissance on the Ten Mile.

One section of contour map covering a part of this region was surveyed last year, and proofs courteously furnished by the Chief Geographer, just as our estimates were being closed. The country is so broken and some of the problems of reservoir site and storage so difficult that an instrumental survey is needed for definite information. I recommend that a complete survey be made over to these tributaries of the Delaware, with a view to the possibility of failure in satisfactory negotiations for the Housatonic and Ten Mile.

Eleventh—LAKE GEORGE.

Lake George has often been suggested. The briefest investigation will show that its watershed is utterly inadequate to supply any such quantity as is needed for New York. The watershed is so small and the evaporating surface so large, that not more than 150 million gallons per day in a dry year could be relied upon from this source, or no more than from the Ten Mile alone. Its elevation, too, is about 222 feet lower than the Hudson at the point of diversion at Corinth proposed for the Adirondack supply, and would not flow to New York, without pumping, at an elevation sufficient for use. Lake George should be dismissed from consideration as among the manifest impossibilities.

Twelfth—THE GREAT LAKES.

The Great Lakes have sometimes been spoken of as the source to which the Greater New York of the distant future must go for an adequate supply. A brief inspection of the map will clearly indicate that the distance to Ontario, the nearest, is a barrier which is insurmountable in view of financial considerations, while the elevation of the source would necessitate pumping. Comparing this even with the enormously expensive Adirondack supply, we see at once that it is beyond all question inferior and much more expensive.

All of the foregoing sources are indicated upon Map No. 1 accompanying this report.

Summary Regarding Possible Sources.

The reconnaissances and rough estimates of yield and cost for the sources mentioned above, which comprise everything found on the map that gives any reasonable prospect, demonstrates beyond a doubt that for low cost of construction, large yield, safety of dam sites, simplicity of management, low cost of maintenance and supervision the Upper Housatonic is much the most favorable source.

The source that I am led to recommend, after studying all of these possibilities, is the Ten Mile in combination with the Upper Housatonic. No other source possesses equal advantages of nearness, simplicity and economy of operation, and facility for delivery by gravity at an elevation of 300 feet above the sea at the city limits, which would afford a pressure of 100 pounds to the square inch on substantially every fire hydrant in New York City, whenever in the near or distant future one section after another was fitted to receive this higher pressure. This supply would meet all of the requirements for a high-pressure gravity supply that have been so earnestly

Catskill creek and the various streams draining the northern slope of the Catskill Mountains should properly be examined. We made some very brief examination which showed plainly that there would be so much extra expense in the necessary conduit line that the project was less favorable than some of the others; but had not time to pursue the subject. I have been informed that there is more limestone in this region and more probability of hard water than on southern slope of the same mountains, but have no personal knowledge of the accuracy of this statement.

presented by certain prominent underwriting interests and would give a volume of water ample for future growth and development four-fold larger than can be hoped for from the Esopus plus Schoharie Creek, proposed by the Ramapo Company, and at one-seventh the cost per million gallons.

Diversion of Water from Inter-State Rivers.

It would, of course, be impossible for New York, under its own right of eminent domain, to secure any portion of the Housatonic water. While eminent counsel have questioned the possibility, under certain New Jersey decisions, of diverting even the Ten Mile river, *it is a matter of fact and record that such things have been done, and on a very large scale.* The same questions arose in connection with the supply which is now under construction for Boston and the adjacent communities from the Nashua river, and on which, in the State of New Hampshire, there were mill interests much more important than any found along the Housatonic.*

I happened to be a member of the Massachusetts Metropolitan Water Board at the time that these matters were under discussion and for that reason, knowing how easily the questions were solved *in the practical business way*, without recourse to litigation, I have had less hesitation in going ahead and presenting quite a full preliminary estimate upon the Ten Mile and Housatonic project.

In the case of the diversion of the South Branch of the Nashua river for Boston's supply, one side of the case, that adverse to the Board, was very ably and exhaustively reviewed by the eminent lawyers George B. French and Jeremiah Smith, and the result of their researches was published in the Harvard Law Review for November, 1894, under the title of "Power of a State to Divert an Inter-State River." The precedents which they found were perhaps naturally not particularly favorable to the diversion. The actual solution came, not so much as a matter of law as a matter of ordinary business and the ordinary willingness of a man to sell an article in his possession when he can obtain an excellent price and readily procure a substitute.

It is my belief that the advantages of the Housatonic and the Ten Mile supply are so great that the City of New York could, to secure this water, afford to pay every house-owner within the basin to be flooded at Dover Plains, three times the market value of his house, and could further well

* The City of Worcester, Mass., has very recently diverted a tributary of a river which furnishes power to more than a score of Rhode Island factories, and has before Commissioners appointed by the Court, acting iointly for Massachusetts and Rhode Island interests, tried some fifty or more cases of petitions for damages all at one hearing, consecutively; and very promptly reached an award.

afford to pay every owner of a mill site along the Housatonic in Connecticut, three times the real value of the water-power to be taken away, and while New York has other sources to which it can go if a generous offer for these were refused, it is reasonable to expect that the parties owning this property would unite in giving the city an option upon it under a price even less generous than that which I have mentioned.

It would, of course, be dangerous for the city to go ahead and become definitely committed to any such expensive scheme without these rights being made secure in advance.

It is also probable that under proper safeguard of fair recourse in its courts, the State of Connecticut would generously co-operate in furthering a project which would be so much to the material benefit of the neighboring Metropolis and at the same time would be to the great profit of certain of its citizens.

Disturbance of Inhabitants of Reservoir Sites.

The population to be disturbed by such a dam is large, but is, if I mistake not, much smaller in proportion to volume of water acquired than has been disturbed in certain of the sanitary improvements in the Croton Watershed. Although some few would be grieved beyond the power of money to repair, the average man can be assumed to be willing to sell his property for water supply at 50 per cent. above its fair market price, and the city can well afford to pay this, or even more, to secure water from a location so extremely favorable in distance, elevation, simplicity of management and economy of operation.

I have put into my estimate liberal margin for contingencies and for liberal prices, judging by what such things have actually cost elsewhere.

The number of persons to be disturbed is relatively little, if any, greater than by the great reservoir now being constructed for the Boston Metropolitan District, and the disturbance to established industries, factories and the like, along the Ten Mile and Housatonic, is not one-twentieth part so great as that which has been successfully overcome along the South Branch of the Nashua river.

The writer, having been a member of the Massachusetts Metropolitan Water Board while negotiations for hundreds of farms and dwellings were in progress, and having had personal experience that by friendly negotiation three-quarters of all required parcels or properties could be secured at a reasonable moderate advance on ordinary market price, by the Board being friendly and accommodating and ready to take responsibility of settlement

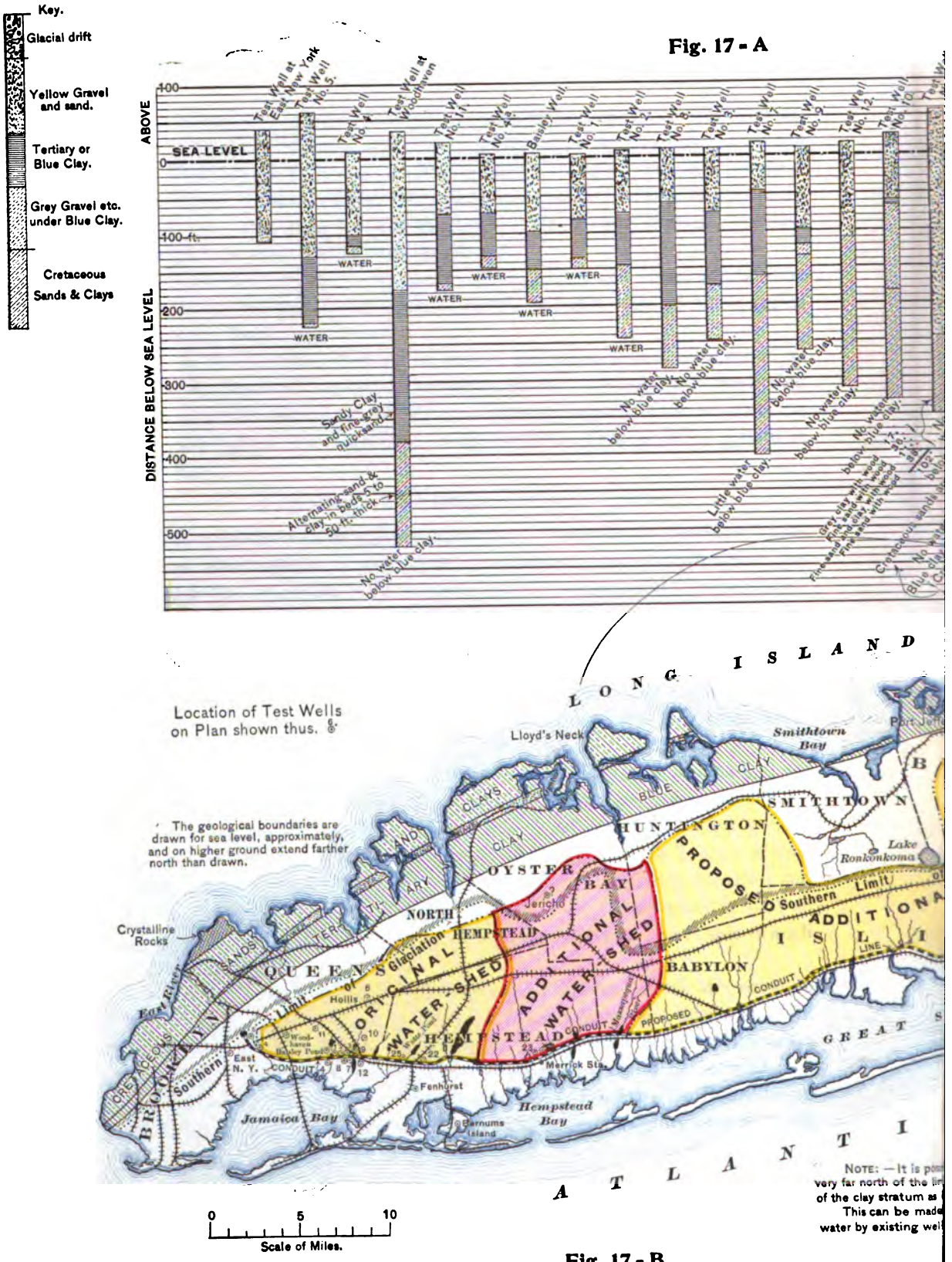
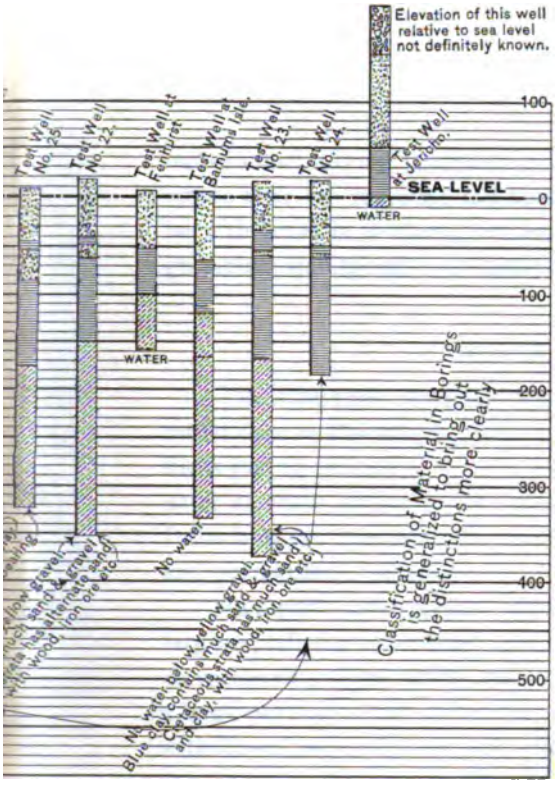
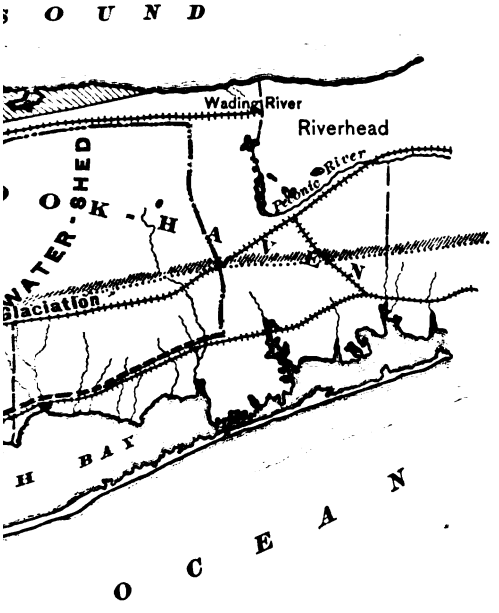


Fig. 17 - A

Fig. 17 - B





e true watershed area does not extend
tion, because of the possible distortion
the section.
testing the level and slope of the ground
ish-drill borings.

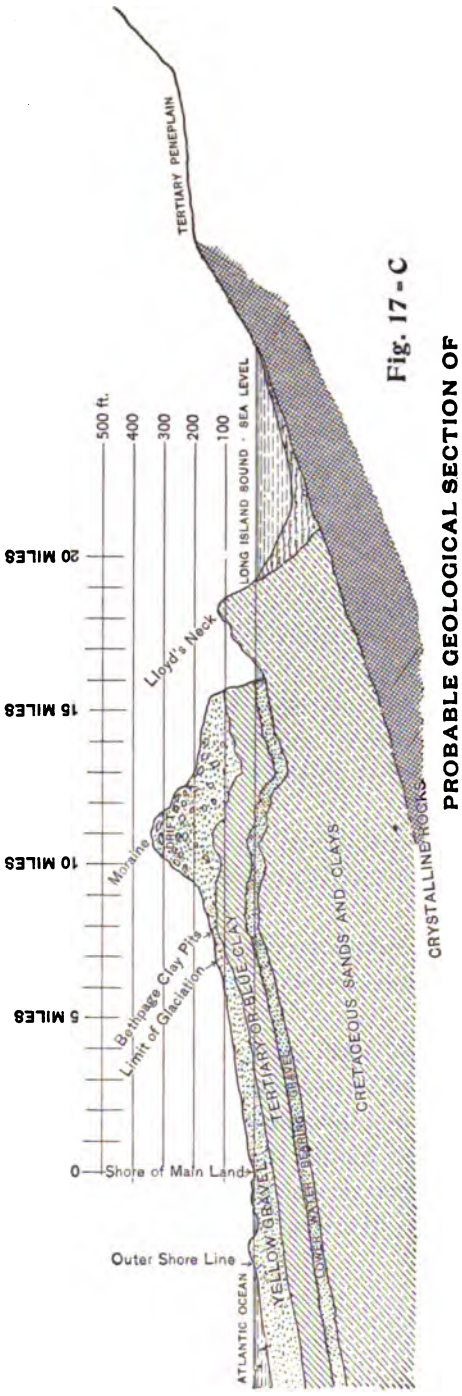


Fig. 17 - C

PROBABLE GEOLOGICAL SECTION OF
LONG ISLAND
FROM NORTH TO SOUTH FROM NEAR LLOYD'S NECK TO NEAR MASSAPEQUA.

out of court, giving prompt payment and extending all possible kindness in ample time for removal, has had less hesitation than might otherwise have been the case in working out the details of a plan so attractive from the engineering standpoint, but causing such great disturbance.

New Structures Proposed.

The structures proposed for the transportation of this water from the outlet dam at Pawling to New York City are clearly indicated in the appendices giving estimates of cost in detail. They are of the same general character as those recently constructed for conveying the waters of the South Branch of the Nashua river toward Boston, and are of the most substantial and permanent character.

As a part of the project and as a safeguard, it is proposed to construct a large equalizing or storage reservoir at Rye Pond, having its surface at an elevation of 355 feet above sea level, but with the aqueducts so arranged that a large portion of this water can be withdrawn for use in emergencies. This withdrawal of surplus storage will conveniently permit of cleaning the aqueduct between Sodom and Rye Pond, by sweeping the interior, at least twice a year.

As a further safeguard, a terminal high pressure distributing reservoir has been proposed on a favorable site at Park Hill, about one-half mile north from the city limits. A careful reconnaissance on the ground by Horace Ropes, C. E., indicates that this structure could be readily made to hold as much water as the large reservoir in Central Park. The cost, as estimated, is to be understood as a mere approximation, but was intended to be safe.

It is to be noted that no safeguard of this kind appears in the proposed plans of the Ramapo Company for its high pressure supply, but is included in the various projects that I have presented.

The cost of water per million gallons by various of the sources described above is summarized in the Table No. 6 already presented, and will be found worked out in detail in the several appendices.

These are to be understood as preliminary approximations, made in nearly every case after a reconnaissance over the ground, and are to be considered merely as preliminary to the work of a careful instrumental survey, with test pits, borings and closer estimates of cost, which should be undertaken, as I believe, vigorously and by several distinct parties, so that all the results should be well in hand before the close of the coming season.

for 100 Million

	Cost for ...lion Gal-...er Day.	Cost per Million Gallons.
	$127,500	$3 49
	56,400	1 55
	42,500	1 16
	.....	.....
	16,800	46
	$243,200	$6 66
...en estimated ...ry much less]		$23 80
...ood Reservoir...		$30 46
...ire 750 million		$8 89
...ated above.....		6 66
Reservoir based on		$15 55

Future Supply for Borough of Richmond.

For Staten Island the outlook for securing good water in adequate quantity upon the island itself is utterly hopeless, and it is plain that the supply which the near future demands must come from the mainland and that it can be most cheaply brought across The Narrows from Brooklyn. This matter will be found discussed in Appendix No. 15.

There are great possibilities of commercial and industrial development along the magnificent water front of Staten Island and ample areas for a large population. Its growth would naturally be greatly facilitated by an assurance of an adequate water supply.

I have not had time to study this problem with so much detail as I would like, but venture to call attention to the fact that the topographical maps indicate that an admirable site for a large destributing reservoir, capable of holding a long supply against the emergency of interruption, can be found between the high hills on the northern end of the island, and that with such a reservoir it would appear prudent from the coast survey charts to lay heavy cast-iron flexible-joint pipes across the channel in two or three lines, a safe distance apart, so that if one should be injured another would be available, and to deliver water through these at a comparatively high velocity into the reservoir above suggested. Careful expert study on the ground, with soundings, and by men who have gained experience on some of the recent large works of laying submerged pipes of this kind, are needed for a precise estimate of cost.

Brief Time for so Broad a Subject.

In closing, it is but fair to state that this report has been written under very great pressure for its early publication, and I must again say that the time at my disposal has been too brief to carry various promising lines of investigation to their proper conclusion. I have had to touch very lightly on certain of the large problems, each of which would afford a field for several months' close study and require instrumental surveys for a more precise answer, but trust that by my fullness of statement I have everywhere made the margin of uncertainty plain and made it easy for the future investigator to review my figures.

I desire to publicly acknowledge the patience and kindness with which Mr. A. Fteley, until recently Chief Engineer of the Aqueduct Commission, and Mr. G. W. Birdsall, Chief Engineer of the Department of Water Supply, have answered my many questions and opened certain records of their offices; to Mr. I. M. de Varona, Chief Engineer of the Brooklyn Works, and to Mr. Alfred Craven, Acting Chief Engineer of the Aqueduct Commission, for facilities afforded for gaugings; to Mr. C. S. Gowen, Resident Engineer

at the New Croton Dam, for aid in various matters; to Mr. E. Sherman Gould, for the interest that he has taken in looking up his notes on the gauging of the Old Croton Aqueduct; to Mr. F. W. Watkins, for the great interest that he has taken in this current-meter work on regauging the flow in the aqueducts.

Mr. Hiram F. Mills has aided me with some very valuable suggestions. I am also indebted to my friends, Mr. F. P. Stearns, Chief Engineer of the Massachusetts Metropolitan Water Board, for many items of data, and to J. Waldo Smith, Civil Engineer, long identified with the East Jersey Works and now Superintendent of the Paterson Water Works, for many valuable suggestions on water waste prevention.

It is but justice to record the zeal and efficiency of those who have assisted me in the more important parts of this work and who have for months, without complaint, devoted not only long days but their evenings and holidays to pushing forward the collection of data and the many long and tedious computations. My acknowledgments are particularly due to Mr. Horace Ropes, whose long experience in very responsible railroad reconnaissance and location in the west has enabled him to carry forward these lines of reconnaissance and estimates with especial promptness, and in whose judgment on costs of grading and tunneling in the country explored, I have confidently relied. To Mr. Edwin D. Pingree, Mechanical Engineer and head draftsman, who has also verified estimates and made independent check computations for many branches of the work and has verified the proof sheets; to Morris Knowles, C. E., formerly office assistant engineer of the Massachusetts Metropolitan Water Board, later for two years resident engineer on the Pittsburgh investigations for improving the water supply, and afterward assistant engineer on the recent investigations for improving the water supply of Philadelphia. Mr. Knowles made most of the field studies on the Housatonic project and supervised the details of the Croton Watershed computations. My grateful acknowledgments are also due to George A. Taber, C. E., recently of Col. George E. Waring's staff in the New York Street Cleaning Department, whose familiarity with the city and the public routine has been of great assistance and who had charge of the measurements of hourly variation in consumption made in the Central Park and the Ridgewood Reservoirs, and who also, with Mr. Knowles, made the surveys in the Ten Mile and Housatonic districts, and to my friend and former classmate, Prof. Crosby, the eminent economic geologist, for the great interest which he has shown in applying the teachings of his science to these questions of water supply.

Respectfully submitted,

JOHN R. FREEMAN,

Consulting Engineer.

Appendix No. 1.

YIELD OF THE CROTON WATERSHED.

To determine the yield of the Croton Watershed requires four distinct series of measurements:

1st. The quantity of water wasted over the dam.
2d. The quantity drawn out through the old aqueduct.
3d. The quantity drawn out through the new aqueduct.
4th. The quantity taken from the natural flow and held back in storage reservoirs, or added to the natural flow by discharge from storage.

I find that the measurements published in the Aqueduct Commissioners' reports, computed in part in the office of the Aqueduct Commission and in part in the Bureau of Water Supply, and commonly accepted heretofore as the basis for all estimates of the daily amount of water that New York City can derive from the Croton Watershed by building more storage reservoirs, average about 10 per cent., or 38 million gallons per day too large, because of the use of data which was less accurate than that which we now possess. As the result of new computations for each day and month of the past thirty-two years, judging of the future by the past, I am led to conclude that the utmost average daily yield which can be relied upon in a series of years of low rainfall after the filling of the great reservoir above the New Croton Dam in 1902, will be 275 million gallons per 24 hours. Should the years 1900 and 1901 have as small rainfall as 1880 and 1881, not over 235 million gallons per day could be furnished continuously in those years, or until the new reservoir is completed.

The cause of the excess of the results of the old computations of actual flow month by month from 1870 to 1894 over those by the new computation, is due to—

1st. The use of a less accurate formula for flow over the dam than we now possess.
2d. The use of assumed co-efficients instead of actual gaugings for computing flow in the Old Aqueduct.

3d. The lack of information which we now possess concerning impairment of carrying capacity of New Aqueduct.

4th. Incomplete allowance in the earlier estimates for the effect of the storage reservoirs.

The reasons for these differences are now so clearly apparent and so firmly established that any competent engineer who carefully examines the records and the premises, will, I believe, be forced to depart from the earlier estimates.

The estimate heretofore accepted of the total run-off month by month from the Croton from 1870 to 1887 is given in Tables Nos. 2 and 3, pages 67 and 68 of the Aqueduct Commissioners' report of 1887, and is described upon page 50 of said book.

On critically examining this estimate and comparing the yield thus found for the Croton with that of certain other streams, and on investigating the formulas used in its computation, various matters to be later described were discovered, which led me to recompute the flow for the whole period during which the records are available, because of the great importance of accurate knowledge of what the river had yielded in the past as a guide to what it may be expected to yield in the future; or as a guide toward answering the questions:

How soon will a new source of supply be needed?

How much more storage reservoir capacity in the Croton Watershed will it pay to provide?

How far will all these reservoirs be refilled from the natural flow in years of low rainfall?

The difference between the earlier estimates and the present estimate is found due mainly to the causes described below:

First Correction—The flow wasting over the Old Croton Dam was overestimated about 9 per cent. by the use of a formula not strictly applicable to this peculiar form of dam, and because of a mistaken assumption in length of overfall. On measuring the length of crest line of dam, I found it shorter than heretofore assumed by 10 feet 6 inches (or 4 per cent.) for all depths less than 8 inches, while the length between wing walls heretofore used was substantially correct. I also found that the dam crest is not absolutely level because of settlement near the centre pier, and that the method of measuring the depth gave results about 0.30 inch too high.

To get a correct measure of the flow at various depths I have had careful experiments made upon a full scale model of this Croton Dam crest by Prof. Gardiner S. Williams at the admirable Hydraulic Laboratory of Cornell University, opened last year.

Second Correction—The flow in the Old Croton Aqueduct at the depth commonly used before the New Aqueduct was opened, is less than previously estimated by about 14,000,000 gallons per day, or 15 per cent. These earlier estimates were based on using for the Old Croton Aqueduct the same co-efficient of flow found for the new, smooth and clean Sudbury Aqueduct, and not upon a gauging of the Old Aqueduct itself.

Third Correction—Gauged by the same observer, with the same instrument, the New Croton Aqueduct is now delivering less water for a given depth of flow than when new, to the extent of about 40,000,000 gallons per day; or, when the depth measured at the head of the Aqueduct is 11.0 feet, the shortage is about 15 per cent.

In the absence of the opportunity to examine the aqueduct tunnel, because, as I am told, water cannot at present be shut off even for a day without interfering with the high-service pump supply, we can from a study of recent depths and gaugings, and from knowledge of what has happened in other aqueducts, feel fairly certain that this impairment of capacity is due to three distinct causes:

(*a*) Serious obstruction at or near Gould's Swamp Siphon.*

(*b*) Serious recent obstruction in the upper five or more miles, such as would be caused by growth of spongilla on the inside surface or a deposit of dead leaves or other rubbish on the bottom.

(*c*) A general coating of the sides and bottom with slime or minute organic growths.

The New Aqueduct has not been emptied for inspection now for four and a half years, and has not, so far as I can learn, been given a cleaning since it was first put in use, nine and one-half years ago. I have little doubt that, with a thorough cleaning, the capacity of this aqueduct would return to the original figures.

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* January 16, 1900—I am informed to-day that on December 29, 1899, water was shut off at the head sufficiently to reduce the depth at head of Gould's Swamp to about 5 feet, and the safety gratings were found badly obstructed by dead leaves and twigs, and were cleaned. This removed the excessive loss of head at this point. The small brick dam across bottom of aqueduct a few feet up stream from the beginning of the Gould's Swamp Siphon was not removed.

Fourth Correction—An error was made five years ago in setting the gauge by which depths in the New Aqueduct are read, so that it makes the depth appear 2½ inches too large; this cause alone contributes about 6,000,000 gallons per day to the over-estimate mentioned above.

Fifth Correction—The effect of storage drawn from Boyd's Corner and Middle Branch Reservoirs and the Croton Lake in modifying the natural flow had not been taken into account in these earlier estimates, neither had due allowance been made for the controlled natural ponds.

Among minor matters aiding to make the new estimate more precise than the old, it may be mentioned that on going to the Croton Dam to consult the original records, I found there was recorded a depth of waste over dam measured each evening, in addition to the morning depth heretofore solely used, thus giving us twice as many separate observations on this variable quantity as had been previously used; and that by a comparison of the keepers' records of depth in aqueduct at Croton and at Tarrytown with those at Sing Sing, we were able to eliminate some small discrepancies and add to the certainty of the data for our estimate.

It is plain that proper care has not been used in the past—and is not used to-day—either in the making or in the preservation of these records. The original records of depths, etc., kept at the Croton Dam prior to July, 1879, could not be found for my examination, and the keeper and the assistant engineer, Mr. Blake, tell me they are probably lost. All of the original record books of the Sing Sing keeper have also been lost, and I was therefore obliged to have recourse to a partial transcript of copies in the office of the Chief Engineer of Water Supply and in the Aqueduct Commissioner's Office. The complete records for North Tarrytown were found on file and proved very useful for comparisons on Old Aqueduct, but the depths given for the New Aqueduct in this Tarrytown record appear very carelessly taken, judging by frequent failure to notice important changes made in depth at the head. This may result from the greater distance of the Pocantico Station from the keeper's headquarters. The measuring station for the Old Aqueduct was close at hand to the Tarrytown keeper, and his observations on this appear more reliable. The record of depth in the Old Aqueduct since 1890 often fails to indicate whether the depth is of flowing water or is a deceptive level due to shutting the gate at Tarrytown, or due to building a little dam inside the aqueduct for some temporary purpose, but it has been easy to get the facts in these cases by questioning the employees.

This matter of a correct estimate of the yield of the Croton Watershed is so important as a basis for measuring New York's present means of water supply, and of such interest to the engineering profession as a guide to the yield of other streams, that the evidence on these matters will be presented here with such fullness as to give the means for its critical examination.

WASTE OVER CROTON DAM.

All previous estimates of the flow over the Croton Dam made by the Aqueduct Commissioners or in the Department of Public Works have been based upon the measurement at 5 o'clock each morning of the depth of water above lip of dam. There are no noteworthy mills on the stream to interfere with the regularity of flow, and, therefore, one depth measurement a day serves very much better than it would on most streams. The depth is measured commonly to the nearest inch, and the present keeper and his assistant tell me that within their experience it has always been obtained by measuring down from the corner of the coping stone of the wing wall on the east bank of the stream at a point just downstream from the intake of the Old Aqueduct and about 120 feet upstream from the crest. This stone has been taken to be 6 feet above the crest and the crest has always been assumed to be level and 250 feet long.

Measurements made about November 1, 1899, shows the actual elevations and length to be as follows, starting from an adjacent and reliable bench-mark of the engineers of the Aqueduct Commissioners:

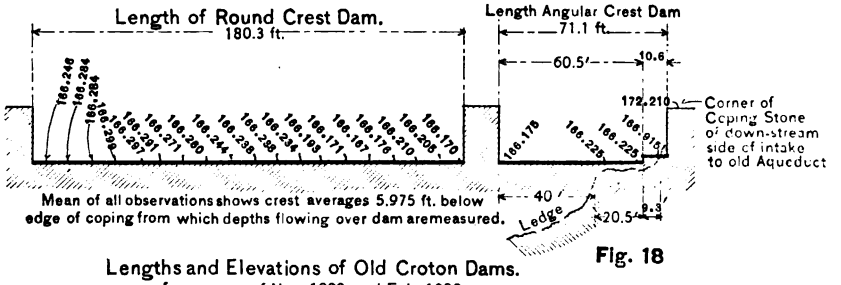


Fig. 18

Lengths and Elevations of Old Croton Dams.
from survey of Nov. 1899. and Feb. 1900
Levels on Round Crest were taken at points 3ft. apart under direction of C. S. Gowen, Div. Eng'r Aq. Com.

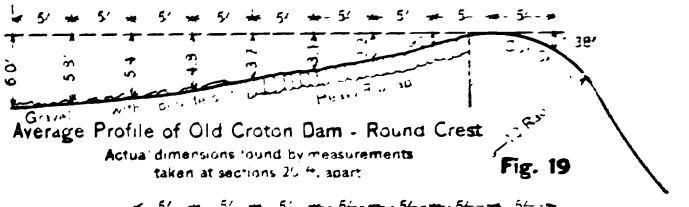
By these levels the elevation of coping stone edge is.....	172.210
Average elevation of crest of dam is.....	166.235
Distance of average crest below edge of coping.....	5.975

These heights indicate that 0.025 feet should be deducted from the depth as now measured from the edge of coping stone, or, in other words, that the real depth averages about 3/10 inch less than recorded by the keeper. It appears fair to assume that in 1868, or about twenty-five years after building, the dam had reached the degree of settlement which now exists. While the observations are commonly made only to the nearest inch, small errors of observation will probably balance in hundreds of observations, and it is important to avoid a constant error of even this small amount—because with the great length of overfall, it cuts a much larger figure in the final estimate than would at first be supposed. Under average conditions this error of 0.3 inch exaggerates the waste over dam about 8 or 10 million gallons per day for five months in the year. I have, therefore, deducted 0.3 inch from the mean of the depths over the dam recorded by the keeper for all days since 1868 in which there was no timber "flashboard" set upon the stone crest.

On February 13, 1900, the old timbers formerly bolted to the stone crests were shown me by Mr. Twigger, Keeper's Assistant, and I found the corners slightly worn, but the general shape well preserved. They were found lying on the margin of the lake about 200 feet north from the dam, and at that time were wet from several days of soaking rains and fog, and were thus probably of nearly the same diameter as when in use. The thickness or depth was found to vary from $11\frac{1}{2}$ to $11\frac{7}{8}$ inches, averaging 11.7 inches. Making a small allowance on the one hand for shrinkage, and on the other for the slight rounding of the corners and for the 0.3 inch average error in elevation of gauge bench as compared with the stone crest, noted above, it appears that the depth of water flowing over the dam when the timber was in use, is probably correct as it stands in the keeper's record.

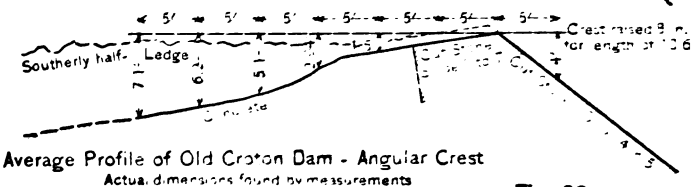
The keeper, Mr. Holbrook, and his assistant, Mr. Twigger, both assure me that in the sheltered location where the depth is measured, the wind very rarely causes waves of such height as to interfere materially with reading the depth washing over the dam to the nearest inch with certainty.

The two parts of the dam are of very different shape of crest, as shown below.



Average Profile of Old Croton Dam - Round Crest
Actual dimensions found by measurements taken at sections 2½ ft. apart

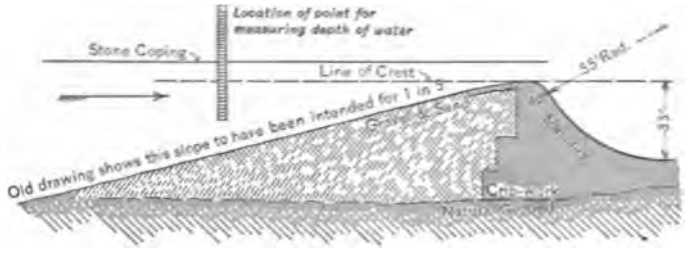
Fig. 19



Average Profile of Old Croton Dam - Angular Crest
Actual dimensions found by measurements taken at sections 2½ ft. apart

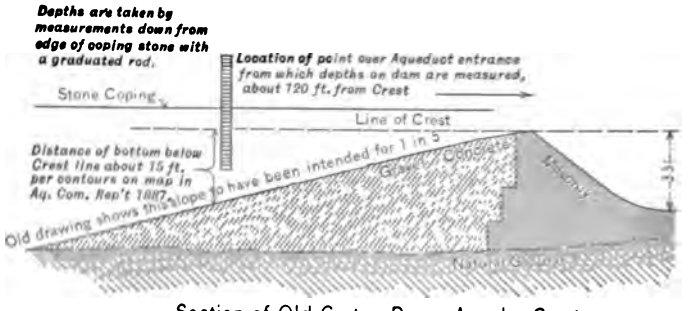
Fig. 20

These sketches are from the average of careful measurements and levels made at sections about 20 feet apart by G. A. Taber, C. E., on October 15, 1899, under my supervision. It will be noted that the upstream slope is hollowed out as compared with the original design.


Location of point for measuring depth of water
Stone Coping
Line of Crest
55' Rad.
Old drawing shows this slope to have been intended for 1 in 5
Masonry
Concrete
Natural Ground

Section of Old Croton Dam - Round Crest
From Wegman and Aq. Com. Rep't, 1887

Fig. 21


Depths are taken by measurements down from edge of coping stone with a graduated rod.
Location of point over Aqueduct entrance from which depths on dam are measured, about 720 ft. from Crest
Stone Coping
Line of Crest
Distance of bottom below Crest line about 15 ft., per contours on map in Aq. Com. Rep't 1887
Old drawing shows this slope to have been intended for 1 in 5
Masonry
Concrete
Natural Ground

Section of Old Croton Dam - Angular Crest
From Wegman and Aq. Com. Rep't, 1887

Fig. 22

ANGULAR CREST SECTION OF OLD CROTON DAM.

Fig. 23

VIEW LOOKING SOUTH, EXHIBITING "STEP" AND OBSTRUCTION BY LEDGE AT SOUTH END.

Fig. 24

VIEW LOOKING SOUTH, EXHIBITING ROUGHNESS OF SLOPE APPROACHING CREST, AND EXTENT OF OBSTRUCTION BY LEDGE.

Photographs taken October 18, 1889.

The figures marked in white on stones give the distance from south end of Spillway.

ROUND CREST SECTION OF OLD CROTON DAM.

Fig. 25

VIEW LOOKING NORTH, EXHIBITING ROUGHNESS OF SLOPE APPROACHING CREST

Fig. 26

VIEW LOOKING SOUTH, EXHIBITING ROUGHNESS OF SLOPE APPROACHING CREST.

Photographs taken October 18, 1899.

The figures marked in white on stones give the distance from south end of Spillway.

The approach to the southerly or sharp crest section is obstructed by an outcrop of ledge for about 30 feet near the southerly shore and extending irregularly about 40 feet upstream from the crest. This ledge is blasted off at a rough general level from 6 inches to a foot below the water when level with the crest.

From inspection when the water was low and from a study of the contours given in a plate in the Aqueduct Commissioners' report of 1887, which shows 11 feet of channel-depth, below level of crest, opposite to the point where depth of water wasting is measured, there appears to be ample depth of channel so that there can be no error due the head absorbed by the velocity of approach. The gauge which is attached to the stone pier between the two dams at a distance of about 40 feet upstream from the crest (which gauge is now indistinct and said to be seldom or never used), appears to be well back of the influence of surface slope. On February 13, with 3 feet depth of water running over the dam, I found no indication of conditions impairing the accuracy of these depth measurements.

Old Formula for Computing Waste Over Dam.

The formula upon which all tables heretofore used for computing the flow over the Old Croton Dam have been based is

$$Q = 3.4\, L\, H^{\frac{3}{2}}$$

Tracing back the origin of this table and formula, Mr. G. W. Birdsall found for me an old document said to be in the handwriting of Captain Dresser, made probably about 1870, which is apparently the original computation. On this sheet six different formulas for discharge over a weir had been set down, and although among these was the formula $Q = 3.012\, L\, H^{1.53}$, from page 136 of the Lowell hydraulic experiments of the late James B. Francis, which was the only one out of the six that had been derived from a broad-crest weir with a sloping approach (and which should have suggested that the form of the crest and the suppression of contraction largely modified the discharge), the above formula, given on the authority of Eytelwein, was selected.

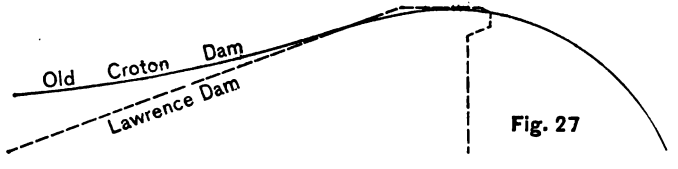


Fig. 27

COMPARISON OF FORM OF CREST OF OLD CROTON (ROUND CREST) DAM WITH THAT OF MODEL OF LAWRENCE DAM. USED IN EXPERIMENTS OF JAS. B. FRANCIS, C. E.

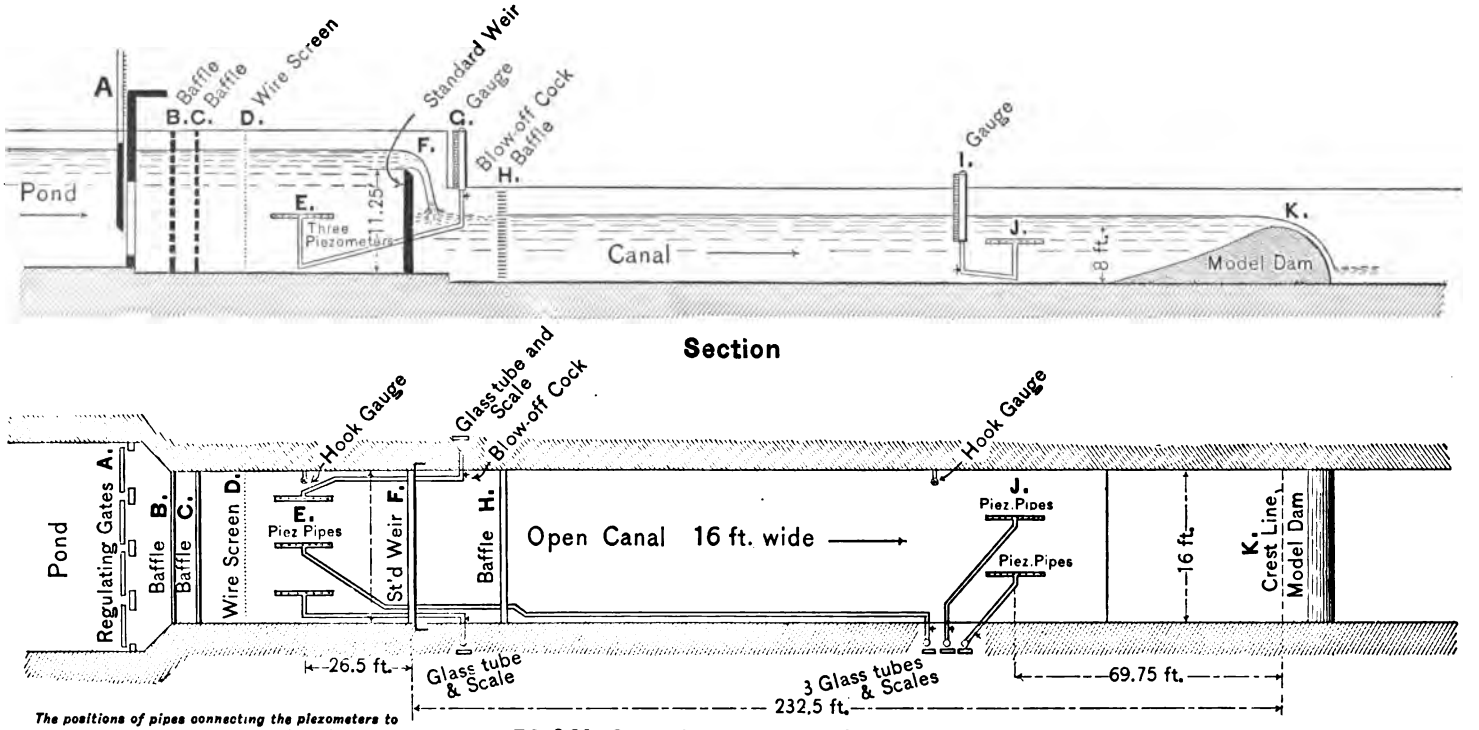
Cornell Experiments.

From superimposing a drawing of Mr. Francis' flat-top model over a drawing of the round Croton crest and noting their general coincidence and that the Francis experiments gave about 12 per cent. less water than the old Eytelwein formula heretofore used for the Croton Dam, and because the experiments in France a few years ago by the distinguished engineer, Henry Bazin (see Annales Ponts et Chaussées 2d, 1898, p. 151), had shown that a great difference in the discharge at a given depth was sometimes produced by seemingly small changes in the form of the crest, and because so much was at stake in this case in knowing with accuracy the quantity which had been wasted over the Croton Dam in past years, and which it is hoped to utilize in future by building storage reservoirs, I ventured to apply for the use of the remarkable facilities found in the Hydraulic Laboratory opened last year at Cornell University, and which had already been put to practical use during the summer, by tests of flow over certain types of dam crests for the United States Deep Water Ways Commission.

Professor Fuertes, Director of the College of Civil Engineering, himself years ago an engineer in charge of a part of the Croton Works, heartily co-operated, and my special thanks are due to him and to Professor Gardiner S. Williams, the engineer in charge of the laboratory, for his earnest personal attention to the construction of the model dams and his vigorous and careful carrying on of the experiments at short notice to the exclusion of all other work and at much personal inconvenience.

The method of experimentation consisted in causing the same water to pass over a standard sharp-crested weir and then over a model dam of the same length. This standard weir was without end contractions, and was strictly comparable with the weirs whose discharge into measuring basins had been accurately measured by Francis, Fteley & Stearns, and Bazin. The depth of this water above the level of the standard weir and its depth above the crest of the model dam were measured with great care.

The following sketches will make the methods and conditions clear:



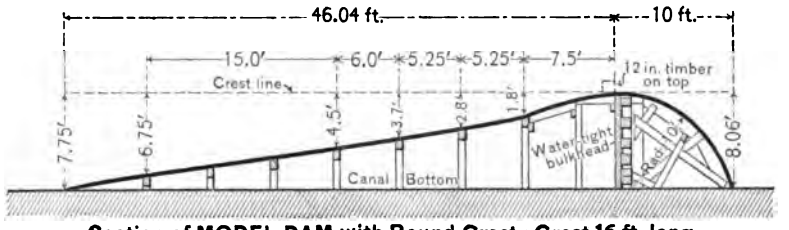
The positions of pipes connecting the piezometers to the glass tube gauges were not exactly as here shown; this drawing being illustrative of the method of connection.

PLAN OF EXPERIMENTAL CANAL

Hydraulic Laboratory of Cornell University.

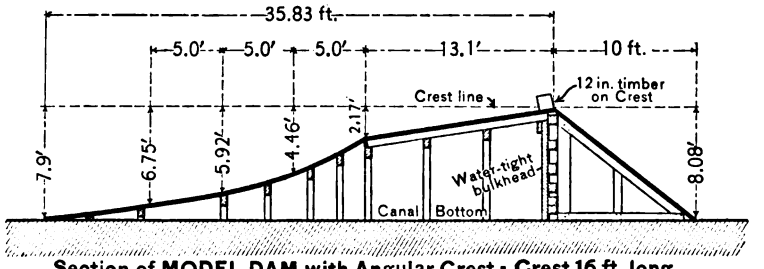
Fig. 28

Not drawn to scale



Section of MODEL DAM with Round Crest · Crest 16 ft. long Fig. 29

After experiment 106, the bottom of model dam was cut off and the crest lowered 2 feet



Section of MODEL DAM with Angular Crest · Crest 16 ft. long Fig. 30

Water from the pond in any desired quantity was admitted through the regulating gates at A. The swirls and eddies were removed by the baffles, B and C, and the screen D, of wire cloth, of about 1-4-inch mesh. The water then, moving smoothly and slowly because of the great depth of the canal, passed over the standard weir F, and then after being quieted by the baffle or screen H, and by passing through the long canal, it passed very smoothly over the crest of the model dam K, and thence ran down the canal to the waste gates. The depths over the standard weir F and over the model dam K were simultaneously measured at half minute intervals by two, and sometimes three, observers, who noted the water level in glass tubes of 1-inch internal diameter, attached to finely graduated scales, firmly set. These glass tubes were connected with piezometer pipes of 1-inch galvanized iron, placed parallel to the current (as shown), perforated with small holes along their sides, the ends being plugged. For a check, the depths were also occasionally measured independently with a hand hook gauge in the open channel. Care was taken that the flow should be well regulated, and the depth steady before each experiment began. Such numerous independent checks on measuring the depth were had in all the later experiments that I am

confident that the error in the mean value of depth over either the standard weir or the model dam could not have exceeded 0.002 foot, except for a very few experiments with the largest flows; this .002 foot including all error of reading, level and adjustment. To make certain that air had free access under the sheet flowing over the standard weir a perforated pipe was laid in the angle just downstream from the crest and connected with a U-tube draft-gauge. The difference in air pressure was found so small that it could hardly be measured.

The two weir basins have sides and bottom of solid concrete masonry; the weir timbers were massive and so well fitted that the leakage was very small. I carefully inspected each weir and weir basin for leakage, first, when the basin was empty, looking for infiltration; and afterward with each basin above the weir filled within 2 inches of the crest and empty below it, I examined the weir bulkheads for leakage. Numerous other leakage tests were made by shutting off water at the head and noting rate of fall of water level in the intermediate basin, 16 ft. x 232 ft., when drawn just below the level of the crest of model dam. A correction for this leakage is introduced into the final computations, but it will be noted that it was so small that it might have been neglected without sensible error.

For a check on the measurement depths and upon leakage and crest elevation, Professor Williams and I kept independent notes during several of the characteristic experiments, and after bringing the several determinations together and studying the possibilities of error, I believe that the resulting curves showing quantity in cubic feet per second per foot in length discharged at various depths up to 2½ feet over the two model dams with round crest and angular crest, in their normal condition, may be regarded, including all questions of piezometers, leakage and calibration of the standard weir, velocity of approach, and of margin of error in the Francis and the Bazin formulas, as a whole as certainly accurate within two per cent. and probably closer. The disagreement in quantity in cubic feet per second between different experiments on the same model at the same depth on different days was commonly much less than one per cent.

The following models representing different actual conditions on the Croton Dam were experimented with:

Fig. 31

Standard Croton Round Crest Dam.

A–1.—First tested with surface of unplanned hemlock plank; again tested with crest covered with smoothly planed pine boards.

A–2.—Tested with that portion upstream from J covered with cleats and fragments of stone to represent, so far as practicable, the actual roughness of the concrete and rip-rap found on inspecting the Croton Dam when pond was drawn low.

A–3.—Slope rough, as just described, and with wire-cloth of about No. 18 wire and ¼-inch mesh, tacked over the cut-stone portion of crest.

Fig. 32

Standard Croton Angular Crest Dam.

B–1.—First tested with surface of fairly regular unplaned hemlock plank.

B–2.—With cleats and fragments of stone upstream from J, representing the actual backfilling of dam.

B–3.—Same as B–2, with addition of wire-cloth over the cut-stone portion of crest from J to K.

In some of the experiments with the angular crest models, a small amount of air persisted under the sheet of water downstream from K, but in the most of the experiments reported there was little or none. The effort was to avoid air under the sheet of water, in the belief that there would be none under actual conditions on the angular Croton Dam. Later, in February, 1900, I had the opportunity to carefully observe the actual dam with about 3 feet of water running over it and saw there was no air under the sheet of water just below the apex. In certain of the experiments a trap-door about 2 feet square was cut in the downstream apron, and so arranged that air could be admitted under the sheet of water flowing. In the few experiments made with, first, no air, and second, with air under the sheet, no noteworthy difference was found, but pressure for time prevented investigating this matter closely, it being of little practical importance in this particular case.

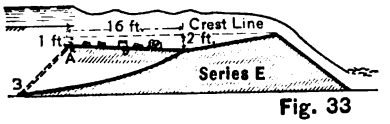


Fig. 33

Round Crest Croton with 12-inch Timber on Top.

C.—Slope roughened as before by fragments of stone and wooden cleats.

D.—*Angular Crest Croton with Timber on Top.*

E.—Angular Crest Croton, without timber on top, with obstruction intended to represent in some degree the ledger existing upstream from northeast end of the Croton Dam. Tested first with end A open, forming a sharp contraction.

E–2.—Same, except that obstruction was provided with sloping approach A–B.

The experiments of Series E were intended to measure the effect of the ledge lying upstream from the southerly end of the angular dam, but they must be taken as illustrative and not quantitative, because this model was not strictly representative of the actual condition of the dam near the ledge, the obstruction upstream from the model being shorter and more regular than the actual ledge. Because of the lateness of the season and the pressure of other work, it was impracticable to take time for constructing a more complete model to represent this ledge; and since the length of the actual crest obstructed by ledge is only about 30 feet, or less than one-eighth part of the length of the entire dam, any lack of precision in the estimate for this portion can have no practical weight in the final result. It appears to the writer that because of the regularity of the upstream edge of this obstruction on the model, the current was deflected upward evenly at A for the entire width of the canal, throwing the obstructions on the platform into an eddy where they produced less turmoil and swirling of the current than would be the case with the actual ledge. Certainly I was surprised at the smoothness of surface current in the Series E-2, and although there was far more disturbance in Series E, it was largely in regular transverse summits and valleys, or waves, which did not necessarily absorb much energy. In both series, E and E-2, the retardation caused by the platform and its obstructions was surprisingly small.

The results of the several experiments at Cornell on these Croton models are given in the following table, No. 13. Each depth given in the table is the mean of from 10 to 20 or more readings of the gauge, and was commonly checked by the observations of two observers reading entirely separate scales connected to independent piezometer pipes, and commonly was further checked by one or two readings by another observer taken in the open channel with a hand hook-gauge. The datum of the scale boards was compared with the elevation of the weir crests, by means of an engineer's wye level, and confirmed in the case of the standard weir by water-levels taken when water was about an inch or two below the crest, by means of simultaneous readings of the glass tube scales and of a small portable hook-gauge clamped to the weir crest.

The experiments are here arranged in order of depth, but it will be noted from the serial numbers that the order of depth was occasionally purposely broken, in order to detect or avoid periodic error. Professor Williams gave his skillful personal supervision constantly, and the writer had the pleasure of spending three days and one night on the ground assisting

in the work and checking up on questions of accuracy. The serial numbers missing from the record are of experiments when the water was in a changing condition or for a period in the early stages when there were no good check observations on the depth over standard weir and when there was probable error from air bubbles in a temporary arrangement of piezometer pipe which was abandoned after experiment No. 8. Had not the work been pressing and the season late and these experiments already delayed by trouble with air in piezometer pipes during the first fortnight's work, I should, as a matter of scientific interest, have developed the curves more completely by observations at shorter intervals.

The observations were plotted on millimetre cross section paper on the large scale of $\frac{1}{2}$ metre vertical = 1 foot depth, and 1 metre horizontal = 10 cubic feet per second per foot of crest. Then curves were drawn, coinciding with the averages of the observations for each kind of crest, which curves served for taking off the quantities at frequent intervals of depth without the use of formulas. See Figure 34. I have purposely avoided the use of a formula and present diagrams instead, because of the smaller liability that these will be misapplied.

Diagram No. 34

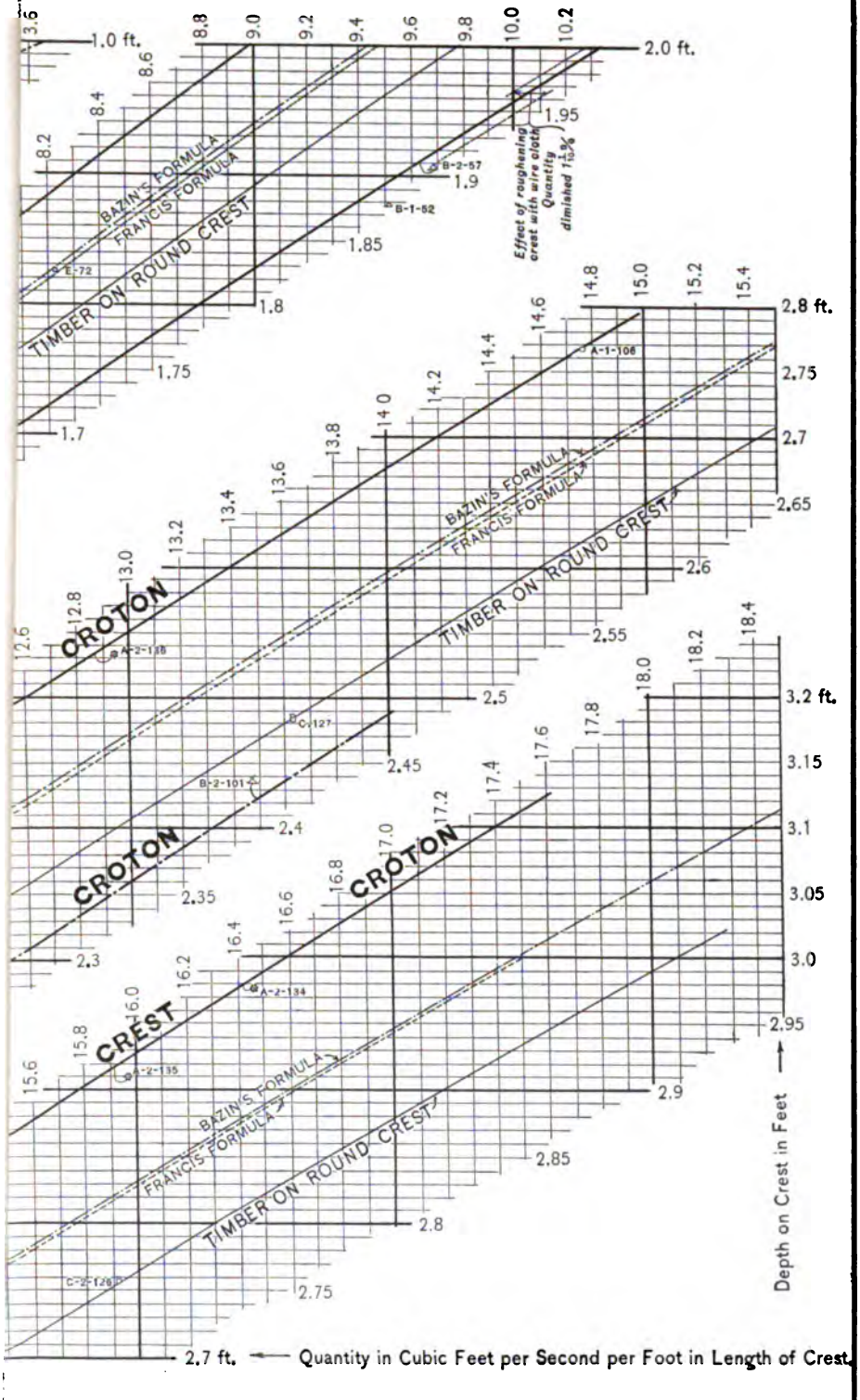


TABLE NO. 13.

Experiments on Quantity of Water Discharged at Various Depths over Models of Croton Dams, made at the Hydraulic Laboratory of Cornell University, November and December, 1899.

DATE, AND SERIES OF EXPERIMENTS. (On November 28, crest of model dam was lowered about 2 feet.)	Serial Number of Experiment.	Observed Depth on Standard Weir. Feet.	Corresponding Quantity in Cubic Feet per Second per Foot in Length over Standard Weir, Including Correction for Velocity of Approach. † Bazin Formula.	§ Francis Formula.	Correction for Excess of Leakage Through Model Dam, per Foot in Length. Cubic Feet per Second.	Observed Depth on Model Dam. Feet.	‡ Mean Velocity in Canal Leading to Model Dam. Feet per Second.	Correction to Depth for Velocity of Approach to Model Dam. Feet.	Corrected Depth over Model Dam. Feet.	Quantity over Model Dam per Foot in Length, Based on Average of Bazin and Francis. C. F. P. S.
November 28, P. M., and 29, P. M. Series A-1. Round crest, smooth pine crest, smooth slope	106	2.6849	14.745	14.783	0.002	2.7229	1.685	0.0439	2.7668	14.762
	108	2.1479	10.581	10.550	0.002	2.1857	1.283	.0259	2.2116	10.562
	109	1.4133	5.600	5.612	0.002	1.4388	0.749	.0087	1.4475	5.604
	107	0.9625	3.167	3.145	0.002	0.9830	0 449	.0031	0.9861	3.154
	110	0.5725	1.468	1.438	0.002	0.5907	0.219	.0008	0.5915	1.451
	111	0.1183	0.163	0.135	0.002	0.1230	0.024	.0000	0.1230	0.147
November 6. Series A-1. Round crest, unplaned plank, smooth slope	1	2.0185	9.572	9.605	.010	2.0897	0.978	.0149	2.1046	9.578
	2	1.7748	7.875	7.911	.010	1.8293	.810	.0102	1.8395	7.883
	3	1.5288	6.267	6.321	.010	1.5878	.661	.0068	1.5946	6.284
	4	1.1841	4.297	4.297	.010	1.2562	.467	.0034	1.2596	4.287
	5	.9335	3.024	3.008	.010	.9929	.338	.0018	.9947	3.006
	7	.5848	1.523	1.486	.010	.6301	.175	.0004	.6305	1.494
	8	.4438	1.018	0.985	.010	.4871	.111	.0002	.4873	0.991

Date, and Series of Experiments. (On November 28, crest of model dam was lowered about 2 feet.)	Serial Number of Experiment	Observed Depth on Standard Weir. Feet.	Corresponding Quantity in Cubic Feet per Second per Foot in Length over Standard Weir, Including Correction for Velocity of Approach. † Bazin Formula.	§ Francis Formula.	Correction for Excess of Leakage Through Model Dam, per Foot in Length. Cubic Feet per Second.	Observed Depth on Model Dam. Feet.	Mean Velocity in Canal Leading to Model Dam. ‡ Feet per Second.	Correction to Depth for Velocity of Approach to Model Dam. Feet.	Corrected Depth over Model Dam. Feet.	Quantity over Model Dam per Foot in Length, Based on Average of Bazin and Francis. C. F. P. S.
December 4, 1899. Series A-2. Round crest, smooth crest, rough slope	134	2.8853	16.467	16.486	.001	2.9227	1.839	.0526	2.9753	16.475
	135	2.8269	15.954	15.985	.001	2.8591	1.794	.0500	2.9091	15.969
	136	2.4617	12.915	12.953	.001	2.4948	1.516	.0357	2.5305	12.933
	137	2.1008	10.170	10.255	.001	2.1420	1.248	.0241	2.1661	10.211
	130	1.5967	6.729	6.753	.001	1.6238	.880	.0120	1.6358	6.740
	138	1.2292	4.548	4.549	.001	1.2597	.623	.0060	1.2657	4.548
	131	1.1125	3.912	3.916	.001	1.1419	.545	.0046	1.1465	3.913
	132	0.6962	1.960	1.933	.001	0.7196	.288	.0013	0.7209	1.945
	139	0.4688	1.107	1.069	.001	0.4873	.166	.0004	0.4877	1.087
November 28, P. M. Series A-3. Round crest, wire cloth on crest, rough slope. *	114	1.9413	9.024	9.056	.003	2.0030	1.174	.0197	2.0187*	9.037
	112	1.3627	5.306	5.316	.003	1.4091	0.712	.0078	1.4129	5.308
	113	0.8302	2.539	2.522	.003	0.8675	.366	.0021	0.8656	2.527
	115	0.4014	0.882	0.847	.003	.4288	.133	.0003	0.4251	0.861
	116	0.1067	0.139	0.116	.003	.1184	.020	.0000	.1144	.124
November 15, P. M. Series B-1. Angular crest, unplaned plank throughout	52	2.007	9.491	9.521	.000	1.8635	0.973	.0147	1.8782	9.506
	48	0.9860	3.279	3.266	.000	0.9246	.370	.0021	0.9267	3.272
	49	.6779	1.884	1.857	.000	.6419	.219	.0008	.6427	1.870
	50	.3619	0.758	0.725	.000	.3481	.090	.0001	.3482	0.741
	51	.1823	.289	.256	.000	.1787	.034	.0000	.1787	.272

November 28. Series B-2. Angular crest, unplaned plank crest, slope roughened with cleats and broken stone	101	2.5297	13.462	13.500	.003	2.4126	1.298	.0262	2.4388	13.478
	102	1.6296	6.934	6.962	.003	1.5251	0.736	.0084	1.5335	6.945
	100	1.0258	3.472	3.466	.003	0.9611	.391	.0024	0.9635	3.466
	103	0.5492	1.392	1.352	.003	.5157	.162	.0004	.5161	1.369
	104	.3251	0.652	0.617	.003	.3051	.077	.0001	.3052	0.631
	105	.0890	.106	.088	.003	.0890	.012	.0000	.0890	.094
November 16. Same crest, similar rough slope	57	2.0318	9.666	9.700	.000	1.8930	0.988	.0151	1.9081	9.683
	53	1.0247	3.470	3.460	.000	0.9605	.391	.0024	0.9629	3.465
	54	0.7478	2.174	2.156	.000	.7028	.251	.0010	.7038	2 165
	55	.4126	0.917	0.883	.000	.3941	.108	.0002	.3943	0.900
	56	.2017	.329	.298	.000	.1952	.039	.0000	.1952	.314
November 16. Series B-3. Angular crest, wire cloth on crest, rough slope. *	61	2.1281	10.358	10.403	.000	2.0053	1.047	.0170	2.0183*	10.385
	58	1.0245	3.467	3.459	.000	0.9787	0.389	.0024	0.9771	3.463
	59	0.7654	2.250	2.232	.000	.7391	.259	.0011	.7362	2.241
	60	.1754	0.275	0.245	.000	.1785	.032	.0000	.1745	0.260
December 1. Series C. 12-inch timber on round crest, rough slope	118	2.0625	9.892	9.922	.003	1.9941	1.037	.0187	2.0128	9.904
	119	1.1684	4.213	4.215	.003	1.1817	0.512	.0040	1.1857	4.211
	120	0.8450	2.605	2.590	.003	0.8832	.328	.0017	0.8849	2.594
	121	.6414	1.742	1.708	.003	68.73	.222	.0008	.6881	1.722
	122	.4631	1.086	1.050	.003	.4986	.141	.0003	.4989	1.065
	123	.2856	0.541	0.508	.003	.2991	.071	.0001	.2993	0.522
	124	.1152	.154	.130	.003	.1177	.019	.0000	.1177	.139
	117	.0701	.077	.062	.003	.0846	.009	.0000	.0846	.067

Date, and Series of Experiments. (On November 28, crest of model dam was lowered about 2 feet.)	Serial Number of Experiment.	Observed Depth on Standard Weir. Feet.	Corresponding Quantity in Cubic Feet per Second per Foot in Length over Standard Weir, Including Correction for Velocity of Approach. † Bazin Formula.	§ Francis Formula.	Correction for Excess of Leakage Through Model Dam, per Foot in Length. Cubic Feet per Second.	Observed Depth on Model Dam. Feet.	Mean Velocity in Canal Leading to Model Dam. ‡ Feet per Second.	Correction to Depth for Velocity of Approach to Model Dam. Feet.	Corrected Depth over Model Dam. Feet.	Quantity over Model Dam per Foot in Length, Based on Average of Bazin and Francis. C.F.P.S.
December 4. Series C (continued). Crest and slope same as December 1	126	2.8210	15.905	15.933	.002	2.7146	1.632	.0414	2.7560	15.917
	127	2.5480	13.612	13.650	.002	2.4519	1.436	.0320	2.4839	13.629
	128	1.5845	6.650	6 673	.002	1.5566	.774	.0093	1.5659	6.660
	129	0.0996	0.124	0.104	.002	0.1046	.016	.0000	0.1046	0.112
	125	.1026	.130	.109	.002	.1070	.0165	.0000	.1070	.118
November 16. Series D. 12-inch timber on angular crest, rough slope	62	1.3141	5.023	5.030	.000	1.2390	.495	.0038	1.2428	5.026
	63	0.8049	2.424	2.407	.000	0.7885	.249	.0010	0.7895	2.415
	64	.4593	1.074	1.034	.000	.4448	.113	.0002	.4450	1.054
November 18. Series E. Obstructed channel with sharp contraction	71	2.3140	11.758	11.797	.000	2.3051	1.154	.0207	2.3258	11.778
	72	1.8224	8 195	8.233	.000	1.8125	0.845	.0111	1.8236	8.214
	73	1.2452	4.632	4.632	.000	1.2278	.507	.0040	1.2318	4.632
	74	0.8765	2.752	2.736	.000	0.8598	.313	.0015	0.8613	2.744
	75	.5872	1.533	1.499	.000	.5745	.179	.0005	.5750	1,516
	76	.3273	0.657	0.626	.000	.3245	.078	.0001	.3246	0.641
	77	.1117	.147	.124	.000	.1120	.017	.0000	.1120	.133
	78	.1136	.153	.127	.000	.1102	.017	.0000	.1102	.140

November 25. Series E (repeated). Obstructed channel with sharp contraction, as above.....	91	1.4747	5.971	5.983	.020	1.4569	.636	.0063	1.4632	5.957
	86	0.9511	3.112	3.093	.020	0 9168	.338	.0018	0.9186	2.982
	90	.7229	2.069	2.045	.020	.6866	.237	.0009	.6875	2.037
	87	.5184	1.280	1.240	.020	.4820	.148	.0003	.4823	1.240
	89	.3127	0.616	0.582	.020	.2811	.071	.0001	.2812	0.579
	88	.1686	.261	.231	.020	.1416	.028	.0000	.1416	.226
November 27. Series E-2. With obstructions rearranged, and with slope instead of sharp edge a up-stream end	97	2.2928	11.596	11.636	.003	2.2927	1.139	.0202	2.3129	‖11.613
	96	2.2919	11.589	11.629	.003	2.2914	1.138	.0202	2.3116	¶11.606
	95	1.1807	4.281	4.282	.003	1.1395	0.473	.0034	1.1429	4.278
	92	1.1473	4.100	4.101	.003	1.1403	.453	.0031	1.1434	‖4.097
	93	1.1355	4.040	4.033	.003	1.1006	.448	.0031	1.1037	¶4.034
	94	1.1514	4.121	4.123	.003	1.1099	.457	.0032	1.1131	4.119
	98	0.4962	1.202	1.164	.003	0.4763	.141	.0003	0.4766	1.180
	99	.0365	0.034	0.023	.003	.0233	.031	.0000	.0233	0.025

* In experiments with wire cloth over crest .004 ft. is deducted from observed depth, to compensate for thickness of wire.

Experiments 9 to 43 excluded because of doubt about air in piezometer—gauges subsequently changed.
Experiments 65 to 70 not of interest and omitted (timber on E).
Experiments 79 to 86 not reported by Prof. Williams.—See his letter of Dec. 11, p. 4.—Head on model uncertain.
Experiments 44 to 47 not of interest—(Apron carried off by current).

† *Bazin's Formula* (English Units).

$$Q = \left(.405 + \frac{.00984}{h}\right)\left[1 + 0.55\left(\frac{h}{p+h}\right)^2\right] lh\sqrt{2gh}$$

l = length of weir.
h = head on weir.
p = depth of channel below crest of weir. = 11.24 feet, for these experiments.

‡ *Depth of Channel of Approach.* (*Feet.*)

	Without Timber on Crest.	With 12-inch Timber on Crest.
For all observations prior to Experiment 106..	7.91	8.91
For all observations after Experiment 105....	6.04	7.04

§ *Francis' Formula* (English Units).

$$Q = (L - 0.1\,nH)\,[3.33\,(H+h)^{\frac{3}{2}} - 3.33\,h^{\frac{3}{2}}]$$

L = length of weir.
H = head on weir.
h = head due velocity of approach.
n = number of end contractions = zero, for these experiments.
‖Trap open. ¶Trap closed.

(See "Lowell Hydraulic Experiments" pp. 252-259 for formula and tables for computing discharge.)

The quantities taken from the curves of Diagram No. 34 were used in constructing a table of the quantities wasting over the actual dam at various depths up to 2.5 feet in height, which was the limit of these experiments. These total quantities at given depths were then again plotted on logarithmic cross section paper and the line of flow produced at the same slope indicated by the experiments up to a height as great as the largest depth ever observed over the dam. This diagram, Figure 35, photo reduced to a smaller scale, is given opposite.

Difference in the degree of roughness of surface of these wide crests was found by the Cornell experiments to make less difference in the discharge of water than might have been expected. Examples of this effect will be found noted on Diagram No. 34. For example, the change from a crest of smoothly planed plank to a crest covered and made rough with wire cloth lessened the discharge 3 per cent. at 0.47 feet depth; 2 per cent. at 0.8 feet depth; 0.8 per cent. at 1.35 feet depth; 1.3 per cent. at 2.1 feet depth.

From personal inspection of smoothness of surface of the stone crests of the dams before and after these Cornell experiments, I was led to adopt a mean between the lines of experiments with the wooden surface and that with the wire cloth over the surface, as fairly representing the actual conditions and for values to be used in our final diagram of discharge over the Croton Dam have adhered to those experiments in which the sloping surface of the model, upstream from the cut stone portion, was roughened with blocks and broken stone.

For the southerly 29.8 feet of angular crest, where obstructed by ledge (including the step), after making a second inspection of the length, position and roughness of the ledge subsequent to the Cornell experiments, I concluded to call the resistance of the ledge, which was actually about 30 feet long, double that found caused on our model E by a more regular obstruction 16 feet long, but used the normal angular crest experiments for the other 40 feet, and of course made due allowance for the decreased depth over the 8-inch step—10.5 feet long—at the southerly end of dam. Possibly this allowance, deducted for effect of ledge, was a little too large. On February 13, 1900, I had the opportunity to inspect this part of the dam while water was flowing over it 3.0 feet in depth, and found sufficient disturbance in the approaching water in the portion above this ledge due to certain wooden steps, direction of approach and other matters, to justify in large

No. 35

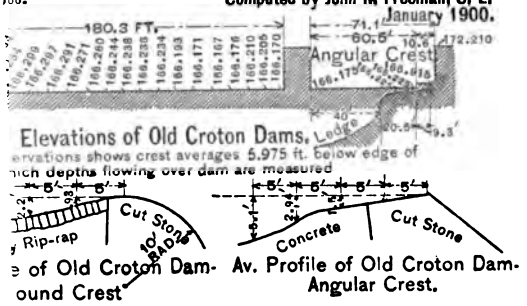
EW YORK WATER SUPPLY

TITY OF WATER WASTING OVER OLD CROTON DAM

AT VARIOUS DEPTHS

putation based on Experiments upon full scale odels of Crests of these dams made at the ydraulic Laboratory of Cornell Universitv Nov. & Dec. 1899.

Computed by John R. Freeman, C. E. January 1900.



e of Old Croton Dam- ound Crest

Av. Profile of Old Croton Dam- Angular Crest.

ntity in Million Gallons per 24 hours flowing over Stone Crests.

0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
14	16	18	20	23	25	27	30	32
37	40	43	46	49	52	55	58	61
67	70	74	77	81	84	88	91	95
102	106	110	113	117	121	124	128	132
140	144	148	152	156	160	165	169	173
182	186	191	195	200	204	209	214	218
227	232	237	242	247	252	257	262	267
277	282	287	293	298	304	309	315	321
332	338	343	349	355	361	367	372	378
389	395	401	407	414	420	426	432	438
451	457	463	469	476	482	488	495	501
514	521	528	534	541	547	554	561	567
581	588	595	602	609	616	623	630	636
650	657	664	671	678	686	693	701	708
722	730	737	744	752	759	766	774	781
796	803	810	817	825	832	840	848	856
871	879	886	894	901	909	916	924	933
949	957	964	972	981	989	997	1,004	1,012
1,028	1,036	1,044	1,052	1,060	1,069	1,078	1,086	1,094
1,110	1,119	1,127	1,135	1,143	1,152	1,161	1,170	1,178
1,195	1,204	1,212	1,221	1,230	1,239	1,247	1,256	1,264
1,282	1,291	1,300	1,308	1,317	1,326	1,335	1,344	1,353
1,371	1,380	1,389	1,398	1,407	1,416	1,425	1,435	1,444
1,463	1,472	1,481	1,490	1,499	1,508	1,517	1,526	1,535

Depth in Inches.	Quantity Flowing.	Depth in Inches.	Quantity Flowing.	Depth in Inches.	Quantity Flowing.	Depth in Inches.	Quantity Flowing.
37	2,800	49	4,270	61	5,950	73	7,820
38	2,920	50	4,400	62	6,100	74	7,990
39	3,030	51	4,540	63	6,250	75	8,150
40	3,150	52	4,670	64	6,400	76	8,310
41	3,270	53	4,810	65	6,560	77	8,480
42	3,390	54	4,950	66	6,720	78	8,650
43	3,510	55	5,090	67	6,880	79	8,820
44	3,630	56	5,230	68	7,030	80	9,000
45	3,760	57	5,370	69	7,180	81	9,170
46	3,890	58	5,520	70	7,340	82	9,340
47	4,020	59	5,660	71	7,500	83	9,500
48	4,140	60	5,800	72	7,660	84	9,680

is diagram pre-supposes depth measured correctly. The dam has ces and average crest is 5.975 feet below edge of stone coping, so method of measurement (Oct. and Dec., 1899) makes depth, as eet = 0.30 inch too great.

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degree the foregoing allowance for retardation; but this disturbed portion being less than one-eighth part of entire length, the greatest possible margin of uncertainty in the flow over it cannot affect the entire discharge over the dam by more than one-half of one per cent.

Estimate of Flow Through the Old Croton Aqueduct.

Examination of reports, careful inquiries in the office of Chief Engineer of the Department of Water Supply and in the office of the Chief Engineer of the Aqueduct Commission, also inquiries of several engineers formerly connected with the operation of the Croton Works, have thus far failed to bring to light any gaugings of the flow in the Old Aqueduct, except the two following:

First—THE TRACY AND CHURCH GAUGING.

G. W. Birdsall, Chief Engineer of the Department of Water Supply, kindly found for me a record of a gauging made for former Chief Engineer Tracy by Mr. B. S. Church, in 1871. This record is to be found on page 49 of Mr. Tracy's record book on file in the Department. Mr. Tracy considered his test to show that with a depth of 5 feet 11 inches at Sing Sing, the aqueduct carried 85.8 million gallons per 24 hours. By recomputation I find errors which reduce this to 75.3 million gallons per 24 hours.

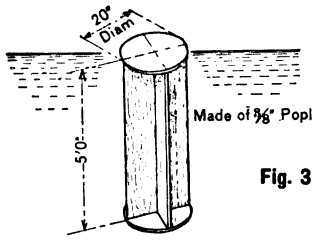


Fig. 36

Isometric Sketch of Float used in Church gauging

Mr. Tracy's data are, in brief, as follows: Four floats of the form shown in margin, Figure 36, made of poplar boards ⅜-inch thick, were sent through the aqueduct from Sing Sing to High Bridge. They were started fifteen minutes apart and the time of passing over this distance of 25.69 miles was as follows:

Float No. 1—Top on Water.	Time of transit,	11	hrs.	29	min.	= 689 min.
" No. 2— " " "	" " "	11	"	29	"	= 689 "
" No. 3—Top 3 inches out.	" " "	11	"	26	"	= 686 "
" No. 4— " 5 " "	" " "	11	"	25	"	= 685 "

Average depth immersed, 4 feet 10 inches.

Average time, 11 hours 27 minutes 15 seconds.

Water was 5 feet 11 inches deep at Sing Sing.
Water was 5 feet 3 inches deep at High Bridge.

Messrs. Tracy and Church computed that the area of section at 5 feet 11 inches depth was 40.25 square feet, that the time of 11 hours 25 minutes occupied in passing 25.69 miles, gave a velocity of 3.3005 feet per second.

40.25 square feet × 3.3005 feet per second × 7.48 gallons per cubic foot × 86,400 seconds in 24 hours, gives 85,841,078 gallons per 24 hours.

The table of flowing capacity of Old Aqueduct used for many years (and still in use, with 10 per cent. reduction), gives 85,197,277 gallons per 24 hours, for this same depth of 5 feet 11 inches at Sing Sing; a coincidence so remarkable as to lead to the conclusion that the table was based on this gauging.

A curve plotted from the depths and quantities of this table used in the Water and Aqueduct Departments is given in Diagram No. 43. The date, origin, authority and computations for this table are stated to be unknown, but Mr. Birdsall identifies the oldest copy in the Department as in the handwriting of Mr. Thompson.

It is plain beyond question that the method of computing this Tracy-Church gauging in 1871 was wrong, but the data are probably all right. The floats were of good form and the data as to depth and time of transit are without doubt accurate, and we can easily apply two corrections that will make the gauging of great value, for it is the only gauging extant made in that decade.

First Correction, Tracy Gauging—It will be noted that the area at only the upstream end of the channel was used by Mr. Tracy, while at the downstream end of the transit the water was 8 inches less deep. The corresponding area is 4.78 square feet, or 12 per cent. less. Obviously, the *mean* velocity of 3.3005 feet per second should be multiplied by the *mean* area, not by the *maximum* area. The straight mean of 5 feet 11 inches and 5 feet 3 inches is 5 feet 7 inches, but observations in later years at various stations along the aqueduct show that the water continues at the same depth from Sing Sing to Tarrytown and falls off but little from Tarrytown to Yonkers, making its chief drop from Kings Bridge to High Bridge in very much the same way that the depth in the New Aqueduct, shown on sheet 40 of Aque-

duct Commissioners' Report of 1895, falls off more sharply near the lower end of the "section not under pressure."

By analogy from various observations, I call the average depth 5 feet 8½ inches for the whole length of run of float. This gives the standard section an area of 38.80 square feet, which should be multiplied by the mean velocity of the four floats, 3.280 feet per second instead of the maximum velocity of 3.3005 found by Tracy.

We find further that 38.80 square feet × 7.48 gallons per foot × 86.400 seconds per twenty-four hours × 3.280 feet velocity, gives 82.2 million gallons per twenty-four hours.

Second Correction—The velocity at the sides and bottom is less than at the central path followed by the floats. A study of the distribution of velocities from the centre to the sides of the Sudbury Conduit with 4.53 feet depth, shows that the mean velocity was only 92.6 per cent. of the velocity in the central portion covered by the cross-section of this float. A similar study of the velocity in different portions of the New Croton Aqueduct at 7 feet depth reported by Mr. Fteley in Aqueduct Commissioners' Report of 1895, sheet 39, after replotting and averaging, snows mean velocity was 92.6 per cent. of the velocity in that central portion of cross-section covered by this float.

On December 16, 1899, we secured by current meter observations the actual distribution of velocities in this Old Croton Aqueduct at Sing Sing for this depth and found that for the area covered by this float, 4 feet 10 inches deep by 20 inches wide, the velocity of water was 3.234 feet per second, while the mean velocity of entire cross-section was 2.962 feet, which is 91.6 per cent. of the velocity in the central area covered by this float.

The average velocity of four floats was 3.280 feet per second, which, multiplied by .916, gives a mean velocity of 3.004 for the entire section and for the distance from Sing Sing to High Bridge.

We, therefore, compute for our final and corrected result—3.004 feet per second × 38.80 square feet = 116.5 cubic feet per second, and 116.5 × 7.48 gallons per cubic foot × 86.400 seconds in 24 hours = 75.3 million gallons per 24 hours, instead of the 85.8 million gallons per 24 hours, found by Tracy and Church.

This 75.3 million gallons corresponds to values of C = 126.6 for R = 2.338 and n = .0138 in the Kutter formula.

The keepers' records for about 300 successive days in spring, summer and fall of 1871 show depth very steady at about 6 feet, averaging exactly 6.02 feet at Sing Sing, 6.02 feet at Tarrytown, and 7.83 feet at Head Gates. I am inclined to believe that the keepers found then, as now, depths an inch or two greater than as measured by the engineers, due to each keeper measuring the depth by plunging a pole to the bottom and including the wash at the top. So, in the diagram, Fig. 43 on page 167, to be used for obtaining the quantity flowing from the keeper's record of depth, I plot the depth for this quantity of 75.3 million gallons at 6 feet 0 inches at Sing Sing, and 7.83 feet at the measuring station about 600 feet below the Old Croton Dam. The corresponding velocity in the standard section from Sing Sing to Tarrytown, which probably has an area of 42 square feet when depth measured at Sing Sing is 6 feet, is 2.804, instead of the 3.004 feet per second found for the entire distance to High Bridge.

The limit of error in this Church-Tracy gauging is probably less than 3 per cent. The possible errors are:

1st. In timing floats. The four floats started 15 minutes apart, were 689, 689, 686 and 685 minutes—maximum difference 0.6 per cent.

2d. The mean depth of water in the aqueduct cannot have been more than 2½ inches in error either way, corresponding to an error in area of not over 3 per cent., and indeed 2 per cent. extreme limit for this is more probable, and the value of coefficient .916 is shown to agree within 1 per cent. of the independent determinations in Sudbury and New Croton Aqueducts.

After a careful study of this matter, I believe that this Tracy-Church gauging, when recomputed as above described, forms the best datum available for computing the amount of water drawn from 1868 to 1875.

Second—THE GOULD GAUGING.

The next gaugings are those made at various dates in the summer of 1884 by E. Sherman Gould, Civil Engineer, then an Assistant Engineer in the Croton Aqueduct Department. I can find no reference to these or to the gaugings by Messrs. Church and Tracy in the reports of the Department of Public Works. Mr. Gould tells me that he devoted considerable time to them, and his reputation is a sufficient guarantee that they were accurately made.

In a letter to me dated November 10, 1899, Mr. Gould states the method and result substantially as follows:

At the time of gauging he found the depth at Sing Sing, 7 feet 4 inches.
Tarrytown, 7 feet 4 inches.
Yonkers, 7 feet.
Kings Bridge, 6 feet 8 inches.
High Bridge, 4 feet 5 inches.

The greater falling off near High Bridge than shown by the Tracy-Church gauging was perhaps occasioned by new connections not existing when that gauging was made and by Central Pork reservoirs being lower at date of Gould gauging. "Plainly," says Mr. Gould, "the only available length for proper measurement with floats was that between Sing Sing and Tarrytown, a distance of almost exactly 5 miles." Floats were made of tin tubes about 1½ inches diameter, of lengths 1 foot, 2 feet, 3 feet, 4 feet, 5 feet and 6 feet; ballasted so they floated upright and immersed to almost the total length. These were let go near the head of the aqueduct and timed to High Bridge and all intermediate points. Neglecting all observations except those at Sing Sing and Tarrytown, because of the varying depth, the following velocities in feet per second were found:

Surface float.....	2.68 feet per second.
Float, 1 ft. deep.....	2.69 " " "
" 2 ft. "	2.74 " " "
" 3 ft. "	2.76 " " "
" 4 ft. "	2.88 " " "
" 6 ft. "	2.93 " " "

Based on these, Mr. Gould took 2.90 as the probable mean velocity in central plane from surface of water to bottom of channel and at first assumed that the mean velocity through the whole mass would be 85 per cent. of this. He would now adopt 92 per cent., thus obtaining 2.67 feet per second mean velocity, and since area of wet section at this depth is 49.19 square feet, Mr. Gould now concludes that the actual flow of the aqueduct at this time, when the depth, 7 feet 4 inches, at Sing Sing, was the greatest that has ever been maintained for a long period, was 84.8 million gallons per twenty-four hours. From later data I figure his observations as making this quantity a trifle less.

For determining the ratio of the mean velocity in the entire aqueduct to the velocity by the floats, we now have the distribution of velocities found in

this aqueduct by the gaugings of October 16, 1899. My own computation of the result of Mr. Gould's several observations is as follows:

TABLE No. 14.

Recomputation of E. Sherman Gould's Gauging of Flow in Old Croton Aqueduct, Made in the Summer of 1884.

Length of Gould's Float.	Per Cent. of Float to Depth at Centre (7.33).	Comparison with Distribution of Velocity Found in Old Aqueduct by F. W. Watkins' Current Meter Gaugings December 16, 1899.				Velocity of Float Noted by Mr. Gould.	Corresp. Mean Velocity in Entire Section, as Deduced by J. F. R.	Corresponding Million Gallons per 24 Hours.
		Proportional Depth in Section Metered December 16, 1899. (See Diagram, page 33).	Mean Vel. in Central Plane for this Depth.	Mean Vel. in Whole Cross Section.	Ratio of Vel. in Whole Section to Velocity in Path of Float.			
Surface.....	1 (?)	Surface....	3.060	2.962	.967	2.68	2 59	82.4
1 foot.....	13.6	0.80 ft.	3.125	2.962	.947	2.69	2.55	81.1
2 feet.....	27.1	1.60	3.170	2.962	.935	2.74	2.56	81.4
3 "	41.0	2.42	3.226	2.962	.918	2.76	2.54	80.8
4 "	54.5	3.21	3.272	2.962	.905	2.88	2.61	83.0
6 "	82.0	4.84	3.220	2.962	.919	2.93	2.69	85.5

Giving the velocity by each of the several floats weight in proportion to its per cent. of depth of immersion, I find, as a mean value for this series of six floats run by Mr. Gould, that the Old Croton Aqueduct then discharged 83.1 million gallons per twenty-four hours, when water at Sing Sing was 7 feet 4 inches deep.

This corresponds to a value in the Kutter formula of C=116.5 for R=2.368 and n=.015.

The limit of error in this Gould gauging appears probably not more than 2 per cent. or 3 per cent. The maximum variation in float velocity from mean is 2½ per cent. The uncertainty in mean area may be 2 per cent., due to uncertainty in mean depth. The coefficient adopted for mean velocity must be correct within less than 1 per cent., or more probably one-half of one per cent.

A study of the daily records of the three aqueduct keepers day by day, during the summer of 1884, shows that for seven months the water in aque-

duct was maintained steadily at the same depth without substantial change. The average of 200 days' observation gives:

Depth below Head Gates.....	9.43 feet.
At Sing Sing.....	7.33 "
At North Tarrytown.....	7.43 "

Mr. Gould's Sing Sing depth was apparently taken at the Gate House and not at Snowdon avenue ventilator. The keeper's measurements of December 16, 1899, show about an inch more depth at the Gate House than at the ventilator. Very probably Mr. Gould, as an engineer, naturally excluded the "wash" on the measuring rod, while the keeper probably included it then, as now. I have taken the gaugings of 83.1 million gallons per twenty-four hours to correspond to a depth of 7.33, as observed and recorded by the keeper at Sing Sing.

The corresponding quantity used in the Aqueduct Commissioners' office in 1887, for computing the flow in 1884 at the same depth was 99,364,000 gallons per twenty-four hours; or *their assumed coefficient* of flow gave 16.2 million gallons per day more than Mr. Gould found *by a series of careful gaugings.*

Third—GAUGING OF OLD AQUEDUCT BY FILLING RESERVOIR.

The next step in the series of gaugings was the discovery reported to me by Mr. Birdsall that when replenishing a reservoir in Central Park, it did not fill so fast as the old table of the flowing capacity mentioned on page 144 called for, and that this rough measurement showed that an amount from 6 per cent. to 10 per cent. must be deducted from the quantities in the table. Mr. Birdsall thinks no record of this gauging of the Old Aqueduct by its discharge into the reservoirs has been preserved. The conditions existing in the City from 1875 until the completion of New Aqueduct would now appear to have made a high degree of accuracy impossible in thus measuring the flow of the aqueduct into the reservoirs, because the City was continually demanding more water than could be delivered, thus not permitting an experiment on a large scale; moreover, shutting off of the Old Aqueduct was very infrequent, and the junctions of the supply with the distribution connections and gates at the Central Park Reservoirs were very complicated, and such as to interfere with an accurate measurement of this kind.

The recent investigations show that even this 10 per cent. deduction was not enough.

Fourth—ESTIMATES BASED ON SUDBURY COEFFICIENT.

The next noteworthy step in computation of flow in Old Aqueduct was the preparation of the estimate of yield of Croton Watershed in Aqueduct Commissioners' Report of 1887, made by W. S. Church. In this, all the gaugings above referred to, made on this very aqueduct, appear strangely disregarded and a fresh start was made, not by a new gauging, but by *assuming that the coefficient of flow of the Old Croton Aqueduct was precisely the same as that* determined a few years before by Fteley and Stearns *for the new and clean Sudbury Aqueduct* of Boston.

Looked at in the light of that time, this may have been natural, for the Sudbury presented the most accurate determination of flow in a large aqueduct that had ever been made. The New Croton Aqueduct had not then been completed and its smaller coefficient of flow discovered. The strange thing is that Mr. Church did not first gauge the flow by a current meter at Sing Sing in 1887, just as we did on December 16, 1899.

Comparison of Old Croton Aqueduct with Sudbury.

In the light of to-day, I consider it absolutely certain that a mistake was made in assuming that the old, uncleaned Croton had so high a coefficient of flow as the new, clean Sudbury, and that this cause alone has led to an overestimate of probably 14 million gallons per day in the yield of the Croton Watershed through the great droughts of 1880 and 1883, and indeed for almost the whole period from 1868 to 1887.

As is well known, the discharge of the New Croton Aqueduct, when new, proved 6 per cent. less than had been expected from the experiments on the Sudbury Aqueduct; and recent information on the effect of a thin coating of organic growth, or slime, in retarding flow, proves conclusively that certain other aqueducts, when not recently cleaned, convey 10 per cent. less water than the same aqueduct will carry after its walls have been brushed clean.

The Sudbury Aqueduct, built in 1877, under J. P. Davis, A. Fteley and F. P. Stearns, engineers who appreciated the great influence of smoothness on delivery, undoubtedly had a smoother interior and a better alignment than the Old Croton, built in 1847, before the experiments of Darcy had brought the very great importance of smoothness to the attention of engineers. Settlements have occurred where the Old Croton Aqueduct runs over embankments, necessitating a hasty patching of the bottom in many places. An examination of the survey made by the Aqueduct Commissioners of the portion about three miles in length between the Old Croton Dam and the

New Croton Dam shows six long stretches aggregating nearly one-half mile in a total length of about three miles, where bottom has had a patch of cement mortar and concrete from 2 to 3 inches thick, and shows that the grade of the bottom frequently rises and falls 3 inches from a straight line or uniform slope.

The Old Croton Aqueduct was worked to its utmost safe capacity from 1875 to 1890, and was so inadequate to supply the demands that there were few opportunities to draw it off for cleaning.* The record shows that the following were the only occasions on which water has been out of the aqueduct for the thirty-two years, January, 1868, to December 31, 1899.

Old Aqueduct not in Use (dates inclusive).

Water out	384	days—December 13, 1898, to December 31, 1899.
" "	706	" —November 7, 1896, to October 13, 1898.
" "	1	day—April 10, 1896.
" "	446	days—July 9, 1894, to September 27, 1895.
" "	1	day—April 19, 1894.
" "	1	" —March 29, 1894.
" "	1	" —March 11, 1894.
" "	1	" —January 28, 1894.
" "	1	" —December 10, 1893.

* January 6, 1900, on asking Mr. Birdsall more specifically if he had ever found any organic growth in the Aqueduct which would fit the description of spongilla, he tells me that he has seen the upper seven miles of the Old Croton Aqueduct largely covered with a deposit of organic matter, large areas of this projecting an inch from the inside of the walls; that half a day after the water was withdrawn this would begin to die, drop off and decay, and give a most offensive odor. That on several occasions he had men scrape this off and sweep the walls with stiff brooms. That he has seen fragments caught at lower screens after water became cold, which indicated that ice-cold water killed the organic growth.

On February 13, 1900, I carefully questioned the keepers at Croton and at Sing Sing about their recollections of deposits of slime.

Mr. Twigger, assistant to the keeper at Croton Dam, who has been employed at that station for about sixteen years, says that prior to the opening of the New Aqueduct, the Old Aqueduct had gone a long time without cleaning; that as near as he can recollect the deposit in the upper two miles averaged fully ¼ inch thick, or possibly a full half inch thick on the side walls below the wate line, and more on the bottom; that in spots of considerable size it was fully two inches thick—was soft and slimy, and occurred mainly in the upstream 2 miles; that they scraped it off the side walls with hoes, then swept the side walls down with brooms, and then flushed out the bottom with a few inches of water by opening the plug-holes at the culverts and pushing the rubbish along the bottom, either to a plug-hole or a ventilator opening.

He recollects a similar thorough scraping and sweeping in 1898, and another about 1895. Says he understands it was the custom whenever aqueduct was emptied for two days, prior to completion of New Aqueduct, to send in a large gang of men to sweep and clean the aqueduct, but these cleanings did not come often. Mr. Holbrook and Mr. Meade confirm this in a general way, but neither of them has been on the work so long as Mr. Twigger, and it will be noted that the Old Aqueduct was not drawn off at all between 1885 and 1890. Mr. Holbrook and Mr. Twigger say that walls of New Aqueduct were found clean when it was drawn off in 1895.

Water out	73	days	—May 20, 1893, to July 31, 1893.
" "	53	"	—September 30, 1892, to November 21, 1892.
" "	38	"	—October 12, 1891, to November 18, 1891.
" "	3	"	—September 12 to 15, 1885.
" "	1.6	"	—November 16 and 17, 1883.
" "	2	"	—November 28 and 29, 1881.
" "	3	"	—May 25 to 27, 1880.
" "	3	"	—May 8 to 11, 1878.
" "	3	"	—May 7 to 10, 1877.
" "	2	"	—November 23 and 24, 1875.
" "	3	"	—November 10 to 12, 1873.
" "	2	"	—September 7 and 8, 1872 (" Cleaning Aqueduct ").
" "	2	"	—April 15 and 16, 1872.
" "	0.7	day	—December 28, 1872.
" "	2.2	days	—December 31 to January 2, 1872–3.
" "	5	"	—November 30 to December 4, 1869.
" "	2	"	—April 4 and 5, 1868.
" "	4	"	—November 10 to 13, 1868.

For substantially all the time from July, 1874, to June, 1890, the water was about 7 feet 6 inches deep at Tarrytown, or within 11 inches of top of arch, and over the high embankments where settlement had occurred, was probably entirely full. In 1868 it ran at about 5 feet 8 inches, and the depth was from time to time increased to 6 feet 4 inches in 1872, and 7 feet in 1873.

WATKINS' GAUGING—OLD AQUEDUCT.

In view of the importance of this question of the rate of flow of the Old Croton Aqueduct to a correct estimate of the water that has actually run off from the Croton Watershed in the past thirty years, and particularly its yield in droughts, Mr. G. W. Birdsall, Chief Engineer of Water Supply, at my urgent request, very kindly had the Old Aqueduct filled and run at depths as measured at Sing Sing, of from 4 feet to 6 feet on December 14, 15 and 16, 1899, and Mr. Alfred Craven, Acting Chief Engineer, very kindly detailed Assistant Engineer F. W. Watkins (who had made all the current meter gaugings on the New Aqueduct in 1892) to gauge its flow. Mr. Birdsall had previously given orders to have all obstructions removed from inside the Aqueduct, and we understand that it had been thoroughly inspected and the walls and invert cleaned since last in use.

I regret not to have had the opportunity to walk through the Aqueduct the entire distance from the head to Tarrytown, to make careful notes on condition of interior surface and obstructions. I did make a hasty personal examination of the walls for a short distance at three points; namely, for 500 feet near the head gates, near the Sing Sing ventilator and at Sing Sing Kill, and found the side walls and the arch free of slime and clean, but with the joints in some cases a little more rough than they would be on new work to-day. The aqueduct had been out of use in supplying the city for a year. The invert had been covered with from 1 to 2 feet in depth of water for supplying the contractors at the new dam and the prison at Sing Sing. I found that somewhere from one-fourth to one-tenth of its submerged area was covered with patches of soft organic growth about $\frac{1}{8}$ inch thick, these patches averaging say 3 or 4 inches in diameter, and there were also some few small fragments of gravel and plaster on the bottom. From what little I saw of it I should expect the coefficient of roughness would be somewhat more than for the Sudbury "when fairly clean." The aqueduct had been empty for a year, except for the foot or two of water in the bottom, and being well ventilated all along, any slime on the walls would naturally have mostly dried and peeled off, even if not brushed off. For some months prior to our gauging there had been a gap cut through the Old Aqueduct on the site of Jerome Park Reservoir to facilitate work with the contractor's railroad, and therefore the water could not be run through the entire length to Central Park. During our gauging the water was wasted through the large drain pipe at Van Cortlandt Park, a point about eighteen or twenty miles downstream from our gauging station at Sing Sing; and it appears certain that no irregularities of flow could have been produced so far upstream as our gauging station, or at Tarrytown five miles below the gauging station, because of the water not passing through the entire length of the Aqueduct. I personally made a careful measurement of depth at Tarrytown Gate-house and at Sing Sing, while the gaugings at 4.61 feet depth were in progress, and found the depths and slope substantially normal.

Current Meter.

The gauging was made with the current meter of Mr. A. Fteley, Chief Engineer of Aqueduct Commissioners, the same instrument shown on plate No. 47 of Aqueduct Commissioners' report of 1895. Its rate was carefully determined November, 1899, about three weeks before these gaugings, by an assistant engineer of the Metropolitan Water Board at the station used

for rating the Sudbury and Wachusett current meter, by moving it through still water at a variety of speeds. The 58 observations made in rating it fall well in line, the most divergent within the limits of the Croton velocities being not more than one-third of 1 per cent. away from the mean line, while the general coincidence is much closer.

The same current meter was rated carefully prior to the original gaugings in 1891, and again rated at their close in 1893, and possibly at intervening times. The rating in 1893 was by Mr. R. A. Hale at Lawrence, Mass.

Selecting for purpose of comparison a point in these rate curves which fairly represents the actual average condition under which the current meter was used, for example, with the meter indicating 3.500 rev. per second, the several ratings compare as follows:

Dec. 14, 1891, velocity by meter, 3.500 rev. per sec., act. vel..	3.00	ft. p. s.
June 13, 1893, after slight injury, 3.500 rev. per sec., act. vel..	2.89	ft. p. s.
June 28, 1893, after readjustment, 3.500 rev. per sec., act. vel..	3.03	ft. p. s.
Nov. 16, 1899, velocity by meter, 3.500 rev. per sec., act. vel..	2.950	ft. p. s.

For 3.500 revolutions actual distance, mean of above = 2.97 feet.

Since each series of experiments was reduced by the rating made at about the same time, and considering that these different ratings quoted above were made by different observers at different localities, we appear justified in regarding the error in rate of meter as safely inside one per cent.

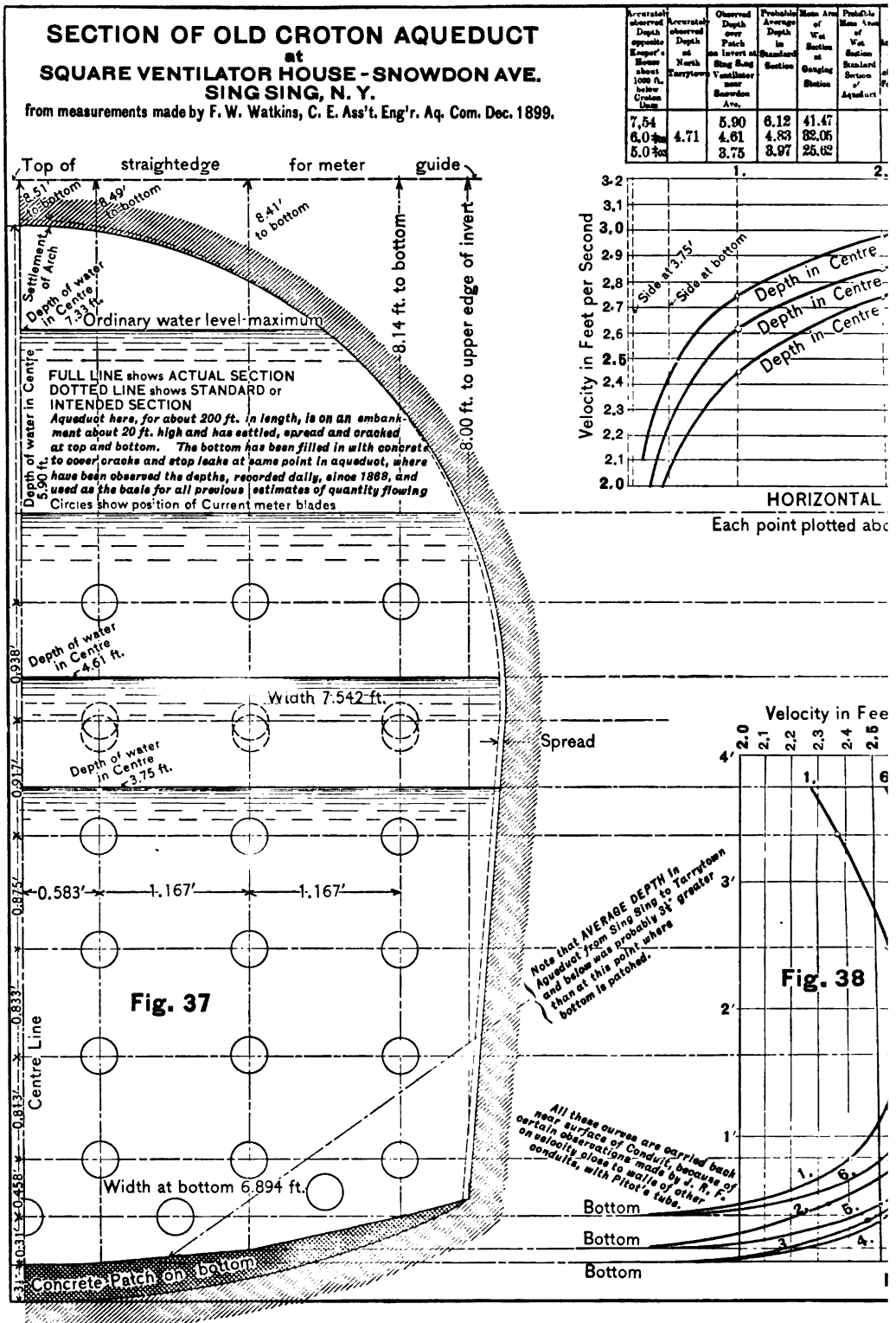
Gauging Station, Old Aqueduct.

The Sing Sing ventilator building near Snowdon avenue was the location selected by me for gauging the Old Aqueduct in 1899, mainly because the gap 6 feet long in arch over Aqueduct at this point gave a convenient place to introduce the meter, and also because this was stated by the Aqueduct Keeper at Sing Sing to be the place where the "Sing Sing depth" has always been measured, and because this depth at Sing Sing has always been the standard for estimating the volume of water flowing. On account of the patch on the bottom at this point, it would be better for any future work to construct a gauging station at some point nearby where the depth is exactly normal.

Prior to the gauging, the small recesses in the side walls of aqueduct at this locality were filled flush with brick laid in cement. The method followed in gauging was very much the same as that described for the New

SECTION OF OLD CROTON AQUEDUCT
at
SQUARE VENTILATOR HOUSE - SNOWDON AVE.
SING SING, N. Y.
from measurements made by F. W. Watkins, C. E. Ass't. Eng'r. Aq. Com. Dec. 1899.

Accurately observed Depth opposite Keeper's House about 1000 ft. below Croton Dam	Accurately observed Depth at North Tarrytown	Observed Depth over Patch on Invert at Sing Sing Ventilator near Snowdon Ave.	Probable Average Depth in Standard Section	Mean Area of Wet Section at Gauging Station	Probable Mean Area of Wet Section Standard Section of Aqueduct
7.54		5.90	6.12	41.47	
6.0 ±	4.71	4.61	4.83	32.05	
5.0 ±		3.75	3.97	25.62	


Top of straightedge for meter guide
8.51' to bottom
8.49' to bottom
8.41' to bottom
8.14 ft. to bottom
8.00 ft. to upper edge of invert
Settlement of Arch
Depth of water in Centre 1.33 ft.
Ordinary water level-maximum
Depth of water in Centre 5.90 ft.
FULL LINE shows ACTUAL SECTION
DOTTED LINE shows STANDARD or INTENDED SECTION
Aqueduct here, for about 200 ft. in length, is on an embankment about 20 ft. high and has settled, spread and cracked at top and bottom. The bottom has been filled in with concrete to cover cracks and stop leaks at same point in aqueduct, where have been observed the depths, recorded daily, since 1868, and used as the basis for all previous estimates of quantity flowing
Circles show position of Current meter blades
Depth of water in Centre 4.61 ft.
Width 7.542 ft.
Depth of water in Centre 3.75 ft.
Spread
0.938'
0.917'
0.875'
0.833'
0.813'
0.458'
0.31'
0.583'
1.167'
1.167'
Centre Line
Width at bottom 6.894 ft.
Concrete Patch on bottom
Fig. 37
Velocity in Feet per Second
Side at 3.75'
Side at bottom
Depth in Centre
Depth in Centre
Depth in Centre
HORIZONTAL
Each point plotted abc
Velocity in Fee
Note that AVERAGE DEPTH in Aqueduct from Sing Sing to Tarrytown and below was probably 3' greater than at this point where bottom is patched.
All these curves are carried back near surface of Conduit, because of certain observations made by J. R. F. on velocity close to walls of other conduits, with Pitot's tube.
Bottom
Bottom
Bottom
Fig. 38

	Error found by Approximate method of Arithmetical Average	Mean Velocity found by Integrating these Curves Feet per Sec.	Corresponding Quantity flowing Cubic ft. per Sec.	Million Gallons per 24 hours with Aqueduct clean as on Dec. 15-16 '99	Probable flow for same depth with Aqueduct in ordinary Condition Gauging of Dec. 1899 less 5 % Mil. gals. per 24 hrs.
5	1.4%	2.902	122.8	79.4	75.4
4	1.4%	2.883	90.7	58.6	55.7
1	1.3%	2.697	69.1	44.7	42.5

GAUGING OF FLOW IN OLD CROTON AQUEDUCT
December 15-16, 1899.

NOTE - At this time the side walls of Aqueduct were said to be clean and free from slime. The Aqueduct had been out of use for nearly a year, but with about 1 foot depth of water left flowing in bottom for supply of prison at Sing Sing, Etc. Some patches of slime were found on bottom near Head. The small dams in bottom of Aqueduct are said to have been removed prior to this gauging.

Observations by F. W. Watkins. C. E.
Computations " J. R. Freeman, C. E.

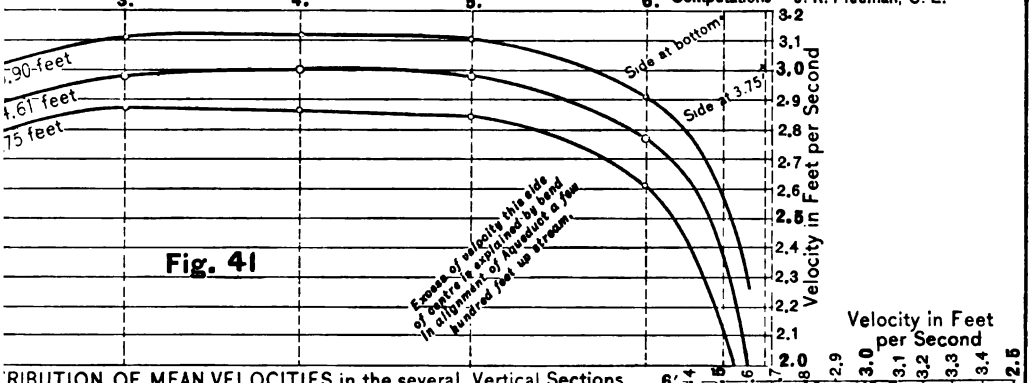

3.
4.
5.
6.
Side at bottom
Side at 3.75
Velocity in Feet per Second
Excess of velocity this side of centre is explained by bend in alignment of Aqueduct a few hundred feet up stream.

Fig. 41

RIBUTION OF MEAN VELOCITIES in the several Vertical Sections
the mean velocity in the vertical plane, according to curves plotted below.

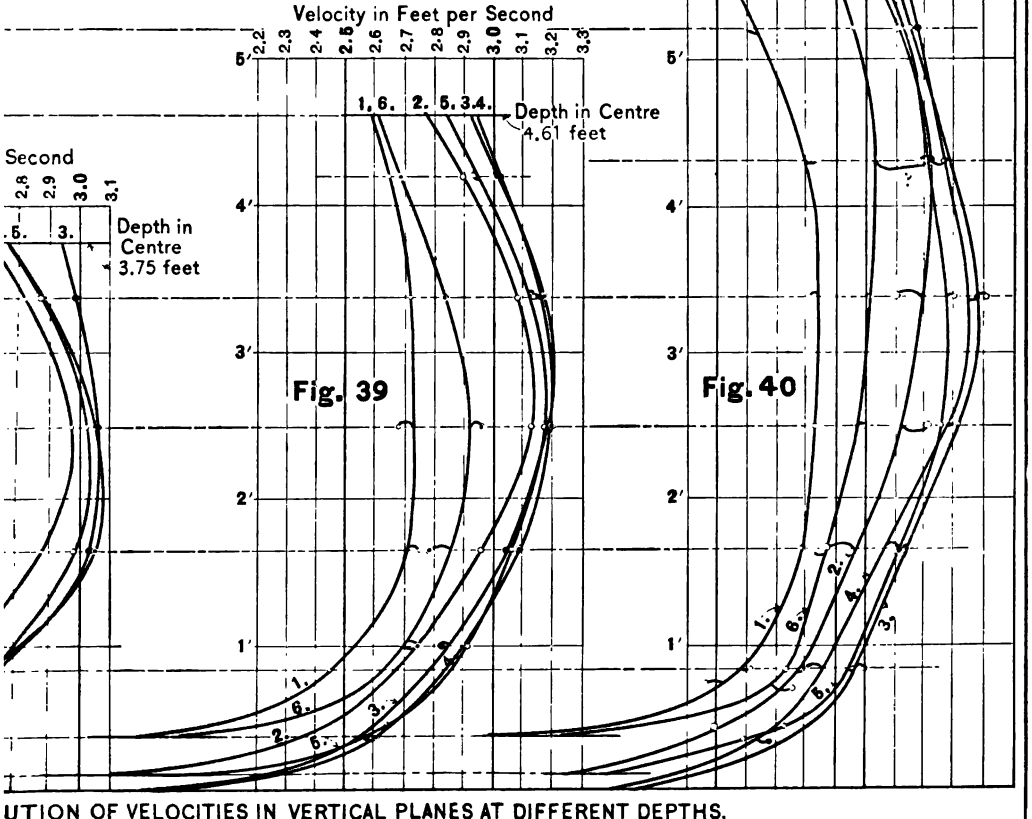


Fig. 39

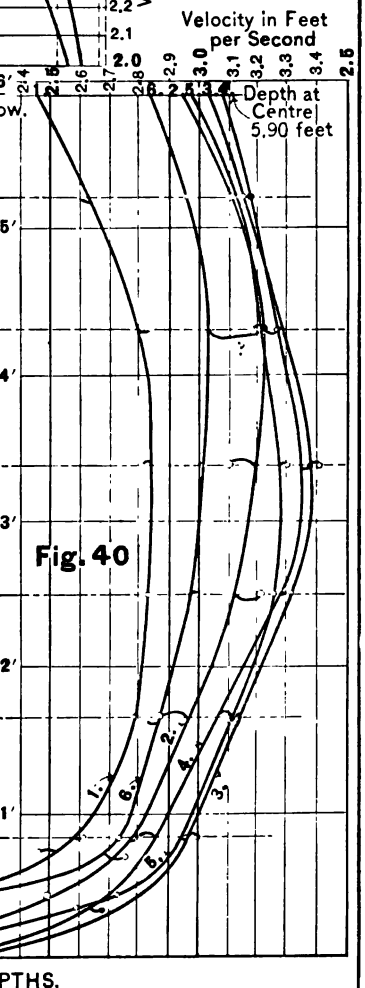


Fig. 40

UTION OF VELOCITIES IN VERTICAL PLANES AT DIFFERENT DEPTHS.

Aqueduct, except that the pendulum-like motion of the meter rod along guide curves could not be followed, because the roof interfered with pivot. The area was divided into "squares" of approximately equal area, and the meter was held in the centre of each square with the register held in gear for exactly one minute and then withdrawn and read. The meter was then put into the next square and the operation of velocity measurement repeated. The mean velocity was integrated by plotting curves and was also checked by taking a direct average of the velocities measured in all the squares. The plottings and the agreement of the mean curves of the several series are all very satisfactory; and the accuracy of the final result in cubic feet per second, or gallons per twenty-four hours flowing in the aqueduct, was apparently made certain within 1 per cent.

Figs. 37–41, inclusive, illustrate the method of measurement and the results. The curves of distribution of velocity are of interest, because of proving the relation of the velocity in the central vein occupied by the floats of Tracy and Gould to the main velocity of the entire section.

Summing up the results briefly, we find:

TABLE NO. 15.

F. W. Watkins' Gauging of Old Aqueduct, Dec. 14, 15 and 16, 1899.

Distribution of Velocity, etc.

Actual Depth in Centre of Section of Meter.	Corresponding Depth as Recorded by Aqueduct Keeper at Sing Sing.	Corresponding Average Depth in Aqueduct Standard Section.	Mean Velocity by Integrating Curves.	Approximate Velocity by Direct Average of Observations.	Cubic Feet per Second from Curves.	Corresponding Million Gallons per 24 Hours, Aqueduct Fairly Clean.	Probable Flow with Walls in Ordinary Condition of Use (less 5%), Mil. Gals.
3.75	3.89 feet.	3.97	2.697	2.721	69.1	44.7	42.5
4.61	4.75 "	4.83	2.833	2.874	90.7	58.6	55.7
5.90	6.04 "	6.12	2.962	2.995	122.8	79.4	75.4

Probable Effect of Slime in Retarding Flow in Old Croton Aqueduct.

The Old Aqueduct was fairly clean at the time of these gaugings by Mr. Watkins in December, 1899, and, therefore, would surely convey more water for a given depth than after being in constant use for a long term without cleaning. Experiments on the aqueduct carrying the water of the Sudbury river to Boston demonstrate that this, when not cleaned for a year, conveys about 10 per cent. less water than it will carry at same depth immediately after having its walls and invert swept clean.

Experiments upon the Wachusett Aqueduct carrying the water of the South Branch of the Nashua river eight miles toward Boston, give substantially the same figure of 10 per cent. as the reduction in delivery at a given depth caused by a year's slime, and recent experiments made upon the New Croton Aqueduct prove that during the 9 1-2 years' constant use without cleaning, since it was first gauged, its carrying capacity has fallen off about 14 per cent. mainly in the upper portion.

Spongilla and algæ of various kinds doubtless breed more rapidly in some fairly clean waters than in others, and they might be expected to breed more rapidly in the Croton water than in the Sudbury water which comes mainly from reservoirs that have had all turf and organic matter removed. I consider that at the most moderate estimate we should deduct from our gaugings of December 14, 15 and 16, 1899, in the Old Aqueduct, at least 5 per cent., to allow for the aqueduct being so much cleaner then than its average condition during the long period prior to 1891 for which we are trying to compute its flow or delivery. At the highest portion of the curve on the diagram of discharge (see Diagram No. 43), I deduct 8 per cent., thus bringing the line down to within 3 per cent. of the Gould gaugings of 1884. I am held back from drawing the line quite as low as the Gould gaugings by the desire to depart no further from the previous published estimates by Mr. W. S. Church in the Aqueduct Commissioners' Report than new, convincing evidence strictly demands, and have gone above the Gould experiments to the full extent that the consideration of the possible error of 3 per cent. stated on page 148 will admit, and it will be seen that in the final curve on Diagram No. 43, this is offset by drawing the line about an equal amount, or to the limit of probable error, below the Tracy and Church gauging. It appears safer to adhere closely to the data thus actually derived from the Old Croton Aqueduct itself *when in the actual customary condition of use*, than to be drawn to one side by the data obtained from other aqueducts with possibly obscure,

but important, differences in the circumstances. The data from other aqueducts, as later appears, are relied upon only for confirmation.

Depths in Old Aqueduct as Recorded by the Keepers.

These depths from the earliest records all along to the present time have been crudely measured, and considering that these form the sole basis of measurement of consumption of water by the City, no suitable or proper facilities have been provided for helping the keepers to accurate measurements. The best facilities are found at the measuring station about 600 feet below Croton Dam, and the height recorded here by the keeper should, according to the means of meaurement provided, ordinarily be correct within an inch, at most. The arrangement here consists of a 3-inch vertical wrought iron pipe, nearly 16 feet long, standing in the middle of the aqueduct, reaching from near the bottom of the aqueduct up through the top of a man-hole cover at the surface of the ground. A 16-foot wooden pole, rather dimly marked in inches, is run down through this pipe until it rests on the bottom of the aqueduct and is then withdrawn and note made of the height to which the water has wet the pole. The object of the wrought-iron pipe is to prevent the current of about 3 feet per second from bending or breaking the pole and to shield the pole from the "wash" of the current which would otherwise force the water up on the upstream side of the pole nearly 2 inches above its true level. I found that when the depth, as carefully measured by the pole slowly lowered into the pipe, was 7 feeet 5½ inches, the depth measured outside the pipe—the true depth—was 7 feet 6½ inches, thus showing what one would expect, that the current past the end of the pipe produces a suction which lowers the level of the water within the pipe nearly an inch. On the other hand, there is a little oscillation within the pipe and the ordinary rapid lowering of the rod into the pipe tends to cause more oscillation, and if the opening at bottom is not very free, the displacement of water by the rod may momentarily raise the water level in the pipe. I, therefore, think it probable that the water mark on the rod will be ordinarily carried up to just about the actual level of the water in the aqueduct. The record is made only to the nearest inch. From studying this record of depth inside aqueduct near its head in comparison with depth in Croton Lake at times in years widely apart when head gates were wide open, I am led to have confidence in the accuracy of the record of depths in head of old aqueduct and had hoped to use them in computing the flow, as a check on the Sing Sing depth, because of their apparent superior precision, but find frequent discrepancies

of 6 inches, or sometimes a foot, between the depth at the head and depth at Sing Sing, after allowing for the difference in grade of aqueduct, which can only be explained by supposing some variable obstruction between the head and Sing Sing; such, for example, as a deposit of dead, water-logged leaves in the slower current of the deep section near the head, or a variable growth of slime or spongilla on its walls. The comparison of depths in old aqueduct at Croton and Sing Sing thus turns out to be more available as a rough indication of cleanness of aqueduct than as a check upon accuracy of measurement. With the aqueduct clean, as on December 16, 1899, the relation of depth at head to depth at Sing Sing appears entirely normal and reasonable, and gives the same coefficient of flow as the downstream portion of the aqueduct after allowing properly for the different grade of bottom of aqueduct near the head.

In this connection it is of special interest to note that the loss of head between the Croton and Sing Sing gauges appears by the records frequently to be 6 inches greater in September and October than in the preceding May and June. This does not occur in every year, and the measurements are not precise enough to warrant any very positive deductions. A plotting of groups of these observations will be found at the bottom of Diagram No. 43. This shows that the Church and also the Gould gaugings found the aqueduct in its ordinary condition of friction loss.

Measurements of Depth in Old Croton Aqueduct at Sing Sing.

The depth at Sing Sing, which has always been the standard for computing the flow, has always been measured, as I am told by the present keeper, "so far as he knows," at the ventilator house near Snowdon avenue. Within this house the aqueduct is open at the top for 6 feet in length and for its entire width. The measurement is made by plunging a stiff pole down into the aqueduct near its middle, then withdrawing it and measuring to the height of the water mark left on the pole. There is no pipe here, as at the head, to avoid the wash of the current or to serve as a guide. By watching the keeper take his measurement and then taking a careful measurement myself, by first running the pole to the bottom and marking its top on the walls of the building and then withdrawing the pole until its bottom just skipped along the surface of the water in and out of the little waves half the time each, and then marking the pole again and measuring between the two marks, thus getting the depth free of disturbance by waves or current, I found the depth 0.14 feet, or $1\frac{3}{4}$ inches, less than

recorded by the keeper. The surface velocity shown by the current meter was subsequently found to be 2.95 feet, for which the hydraulic head is 0.135 feet, thus confirming the extra height or error found to be caused by including "the wash of the current" in the measurement on the pole made in the customary manner. I am credibly informed by certain members of the Engineer Corps that they have noticed other measurements of depth by the keepers along the aqueduct in which an error of from 4 to 6 inches was made by lack of care to place the pole plumb, and in the middle of the invert.

This crudeness and carelessness is to be charged up more against the superiors who fail to provide proper facilities than against men who do not understand the importance of making these measurements with precision.

This Snowdon avenue ventilator is at the middle of an embankment about 300 feet long, of about 15 feet height from natural surface to bottom of aqueduct. As at nearly all sections of the Old Aqueduct built on embankments, the aqueduct has settled here, its sides have spread, the arch has settled, and the bottom of aqueduct has cracked and been patched. This patch, as found by Mr. Watkins, by measurements taken in connection with his gauging, is apparently 3½ inches thick at the middle and thus lessens the depth in aqueduct at this point. I have no information as to when this patch was put in, but assume it has been there many years.

A relatively small proportion of the whole aqueduct rests on embankment and the depth in the average standard section will probably be greater than the depth measured by the keeper at this point by the following amount:

Diminution of depth caused by patch on invert.....	3½ inches.
Less assumed settlement of invert, about.....	1 inch.
	2½ inches.
Excess of measurement caused by wash on rod, about.....	1½ inches.
Net excess of average depth in aqueduct near Sing Sing over depth as measured by keeper, about.....	1 inch.

The keeper's record of depth will, because of including the wash on rod, apparently be 0.14 feet greater than the same depth as accurately measured by Mr. Watkins.

Depth in Old Aqueduct at Tarrytown.

This is of interest as a check on the depth recorded at Sing Sing and for showing that for this five miles at least the hydraulic grade follows the grade of the bottom of the aqueduct. The depth recorded three times daily in the record book (as I am told by the keeper, Mr. Hylands), is measured inside the gate-house by running a stiff pole down in the west side of the aqueduct to the edge of the invert at the small recess about 2 or 3 feet upstream from the gate and measuring to the top of the water-mark left on the pole and adding 7 inches to allow for depth of centre of invert below edge of invert. It is said that formerly the depth was measured at the middle of the aqueduct in the same manner, but a plank floor has been laid over the main opening, which now prevents this. I found on removing two of these planks and carefully running a pole to the bottom, marking its top on the gate-house wall and then carefully raising the pole until its bottom just skipped along the surface, half the time in and half out of the little waves, that the depth was 4 feet 8.5 inches. The keeper's record at same time gave 4 feet 10.0 inches, showing an excess of $1\frac{1}{2}$ inches mainly due to wash of waves on the pole raising the water-mark, and partly due probably to the point touched by bottom of rod not being exactly 7 inches above bottom of invert.

In former years when pole was plunged down in the middle, the " wash " due this surface velocity would be about $1\frac{3}{4}$ inches, so on the whole, we conclude that the true average depth in standard section is about $1\frac{1}{2}$ inches less than shown by the keeper's records.

Mr. Hylands, the keeper, tells me there is no patch on bottom of aqueduct at this point and no sign of settlement and that it is without doubt at standard grade, but says that over the high embankment immediately south, the whole aqueduct has settled about 6 inches, or perhaps more, so that when dry at the gate, the water over the high embankment stands half way up to the knees as one walks through.

These questions of exact depth are important for computing flow in this aqueduct with all possible accuracy from the keeper's record and for computing the flow by comparison with the gauging of other aqueducts.

Old Aqueduct Flow Computed from Coefficient Found in Other Aqueducts.

As already stated, the safest values for our use are those derived from the Old Croton itself, but it is desirable to confirm these by the results found in other similar structures, particularly as we are departing from the estimates issued some years ago under the authority of the Aqueduct Commission.

First—COMPARISON WITH NEW CROTON AQUEDUCT.

This must be regarded as probably having a coefficient of flow applicable to the Old Croton with more accuracy than that of any other structure on which data are now available, because it is filled with water from the same source, and therefore subject to the same organic growths and same accumulations of dead leaves. Its smoothness of walls when clean, and therefore its coefficient of flow, is probably as a whole not very different; because, although brick lining laid in the darkness and crowded space of a tunnel cannot be laid quite so smooth and true to line as by daylight in a "cut and cover" aqueduct, the New Croton was built under the supervision of Mr. Fteley and others who were keenly awake to the influence of smoothness on flow, and when first gauged, it was a new structure, free from any defects of alignment, settlements or projecting patches like those in the Old Croton and its line contained far less curvature.

The New Croton was first gauged in October, 1891, and gaugings of flow repeated at intervals until the spring of 1893. I am told that the earliest gauging was made 1½ years after the aqueduct was first put in use (these gaugings are described in Aq. Com. Rept. 1895, p. 94). So far as I can learn, its walls were not scraped and swept just before the gauging, but so far as is now known, they were fairly clean. The gauging was beyond all question very accurate, and the error of the final values adopted for quantity flowing at a given depth was probably less than one per cent.

Second—COMPARISON WITH THE DORCHESTER BAY TUNNEL.

(On the Boston Improved Sewerage Works.)

This comparison is of interest because of the whole interior surface of this tunnel being in contact with the current, since the gaugings on the New Croton, the Sudbury and the Wachusett Aqueducts and our gaugings

of 1899 on the Old Croton, are all with depths relatively only about half as great as that used in the Old Croton all the time from 1875 to 1890.

This Dorchester Bay Tunnel was built under the excellent engineering supervision of Jos. P. Davis, Eliot C. Clark and F. P. Stearns. It is lined with brick with smoothness of joints probably much the same as those on the New Croton, since it was built under the ordinary difficulties of tunnel work and with a rather leaky roof, and therefore doubtless less smooth than the Sudbury.

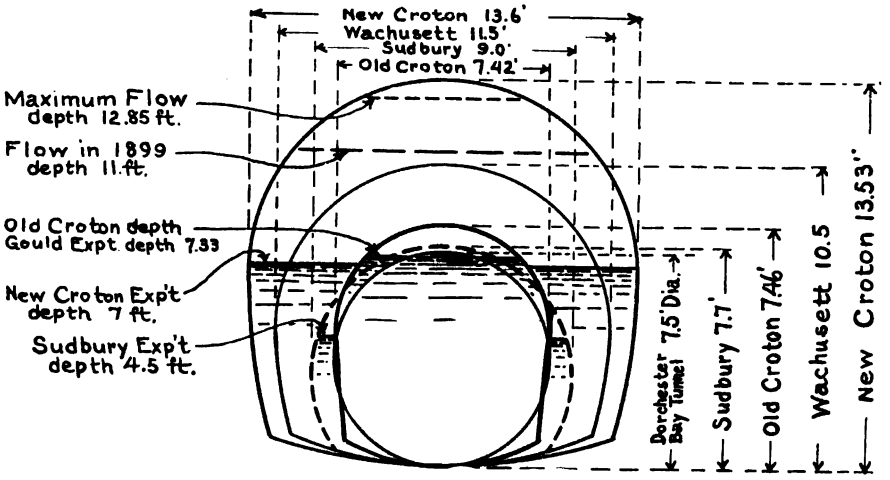
It was gauged some months after having been filled with very dilute, screened sewage moving at slow velocity. This dilute sewage had first passed through large sedimentation galleries.

The tunnel is said to have been flushed out at from 2 to 3 feet velocity once in two weeks, and at the time of the gaugings here used for comparison the sewage was further diluted by three times its volume of salt water from Dorchester Bay. It is therefore probable that the fact that this tunnel carried salt water and sewage, instead of clean, fresh water, did not materially affect its coefficient of flow, and its walls were, I think, probably less coated with slime than those of the upper portions of the Old Croton Aqueduct in 1899. The results are quoted here only for the purpose of comparison, and to illustrate the effect of increasing the proportion of wet perimeter upon coefficient of flow.

Third—COMPARISON WITH THE SUDBURY AQUEDUCT OF BOSTON.

This has already been referred to on page 150. Without doubt this has been the subject of more accurate gauging and testing than any other large aqueduct in the world. It was constructed in the full light of the influence of smoothness on carrying capacity under Mr. Joseph P. Davis, as Chief Engineer, Alphonso Fteley, as Resident Engineer, and Mr. F. P. Stearns, as Assistant Engineer, and has been cared for and operated for many years, and frequently cleaned and regauged, under the supervision of Mr. Desmond Fitzgerald, four of the most eminent hydraulic engineers in this country.

As already stated, the Old Croton has without doubt a smaller coefficient of flow than the Sudbury, but the Sudbury Aqueduct experiments are exceedingly instructive for comparison, particularly the observations made on effect of slime in reducing carrying capacity, which are more fully described on page 175.



Outlines of Aqueducts
Showing comparative water sections

Fig 42.

Fourth—COMPARISON WITH THE WACHUSETT AQUEDUCT.

This is the New Aqueduct of 300 million gallons capacity per day, constructed in 1896–7 for the additional supply of the Boston Metropolitan District. It is lined with brick laid with especial care to obtain smoothness of surface and accuracy of alignment. The final studies for adjustment of observations and determination of its laws of flow have not been made, but I am kindly permitted by Mr. F. P. Stearns, Chief Engineer, to state that its coefficient of flow when new and clean was found to be about 4 per cent. greater than that of the Sudbury; and that after being in use a year without cleaning, the flow for a given depth was found to be about 10 per cent. less than when new; and that on then withdrawing the water and sending men through it armed with stiff brooms and scrapers and brushing its walls and invert fairly clean and then refilling and testing it, the coefficient of flow came back very nearly to the original figure. The coating of organic matter or "slime" on its walls appeared when the water was drawn off to be only about 1-16 of an inch in thickness.

In Figure 42 are outline sketches showing comparative dimensions and water levels for several aqueducts; from which it will be seen that the proportion of wetted perimeter was larger in the Gould experiments on the Old Croton Aqueduct, as commonly used, than in the others, excepting the Dorchester Bay Tunnel, which flowed full.

To compare the coefficient of flow in the Old Croton with that found in these other brick-lined aqueducts, and thus satisfy ourselves of the reasonableness of the values adopted in the final curve of Diagram No. 43, the following comparison is made:

We take, first, the case corresponding to the Tracy gauging, where, by the keeper's record at Sing Sing, depth is....	6.0 ft.
Correcting for effect of "wash" on his measuring rod and for the reduction of depth at the measuring station by the patch on the invert, the corresponding average depth in the standard section of aqueduct is probably.....	6.10 ft.
Standard slope of bottom.....	.00021
Area of standard cross section at 6.10 feet depth.....	41.57 sq. ft.
Wet perimeter	17.78 ft.
Hydraulic mean radius.....	2.338 ft.

We take, second, the case corresponding to the Gould gauging, where, by the keeper's record, the Sing Sing depth

is	7.33	ft.
Correcting as before—mean depth in standard section is	7.43	ft.
Area wet section (per Gould 49.19 for 7.33 depth)	49.72	sq. ft.
Wet perimeter	20.99	ft.
Hydraulic mean radius	2.368	ft.

The values of the hydraulic coefficients found for the corresponding values of R. at the several brick-lined aqueducts above mentioned, together with the flow in Old Croton that these values would give, are as follows:

TABLE NO. 16.

Flow in Old Croton Aqueduct Computed from Hydraulic Factors Determined by Experiment on Other Large Aqueducts. Also Results of Gauging the Old Croton Aqueduct.

AQUEDUCT AND ITS CONDITION. (Slope for Old Croton Aqueduct Averages .00.21.)	Values Found by Experiment at Same Hydraulic Radius (R. = 2.338) as that of Old Croton with 6.00 Feet Depth.		Computed Rate of Flow in Old Croton Aqueduct that Would Result from Use of These Values.		Values Found by Experiment at Same Hydraulic Radius (R. = 2.368) as that of Old Croton with 7.33 Feet Depth.		Computed Rate of Flow in Old Croton Aqueduct that Would Result from Use of These Values.	
	Area Wet Section Old Croton 41.57 sq. ft.				Area Wet Section Old Croton 49.72 sq. ft.			
	C. In Chezy Formula.	N. In Kutter Formula.	Velocity feet per Second.	Million Gallons per 24 Hours.	C. In Chezy Formula.	N. In Kutter Formula.	Velocity feet per Second.	Million Gallons per 24 Hours.
New Croton Aqueduct at various points in its length after 2 years' use, walls probably very nearly clean	131.3	.0133	2.905	78.0	131.4	.0133	2.927	93.1
New Croton Aqueduct near head after 9½ years' use, probably not clean	105.0?	.0166?	2.320	62.3	105.0?	.0166?	2.340	74.4
New Croton Aqueduct near foot after 9½ years' use, probably fairly clean	127.0?	.....	.....	75.3	127.0?	.....	.....	90.7
Dorchester Bay Tunnel (for R. = 1875 ft. C. = 119)	125.0	.0140	2.764	74.2	125.0	.0140	2.784	88.5
Sudbury—New and probably swept clean	139.8	.0125	3.095	83.1	139.8	.0125	3.120	100.2
Sudbury—New special smoothly plastered section, clean	149.0	.....	3.300	84.5	149.0	.....	3.320	106.4
Sudbury—20 years old, walls not clean, flow measured after one year's collection of slime	129.0	.....	.....	76.5	129.0	.....	.....	92.1
Sudbury—20 years old, walls swept clean, one month later than above	143.0	.....	.....	85.6	143.0	.....	.....	103.0
Wachusett—New and clean (approx.)—Wachusett constants	144.9	.....	3.210	86.0	.....	.....	3.230	103.7
Wachusett—After one year's collection of slime From preliminary determinations	.....	.....	2.879	77.3	.....	.....	2.897	92.7
Wachusett—Again scrubbed approximately clean	.....	.....	3.144	84.7	.....	.....	2.941	101.2
Old Croton—Church and Tracy gauging, 18.1—not clean	126.6	.0138	2.805	75.3	.....	.....	.....	.....
Old Croton—Gould gauging, 1884—not clean	.....	.....	.....	.....	116.5	.....	2.60	83.1
Old Croton—Watkins-Freeman gauging, December, 1899—clean	**133.4**	**.0133**	**2.958**	**79.4**	.....	.....	.....	.....
Old Croton—By curve finally adopted for ordinary condition Aqueduct not clean	**123.8**	.....	.....	**73.3**	**118.2**	.....	.....	**85.6**

Conclusions: Flow in Old Croton Aqueduct.

After a careful study of the foregoing comparisons with the flow measured in other aqueducts and after reviewing the above-described gaugings made on this aqueduct by Church, Gould and Watkins, I present the curve of discharge shown in Diagram No. 43 with great confidence that in connection with the records of depth from 1868 to 1891 it gives a correct average measure of the flow in the Old Croton Aqueduct under the actual conditions of use, and that the precision of this measurement is fully equal to that ordinarily obtained in studies of watershed yield. I have tried with but little success to find in the records some safe indication as to the periods when specially clean and when obstructed, and again visited the station at Sing Sing and Croton on February 13, 1900, to question the keepers more specifically upon this. At first I hoped that the difference between the gauge readings at Croton and those at Sing Sing would furnish a satisfactory indication of the condition of cleanliness of the walls, but the lack of precision of measurement prevents it being satisfactory, although it serves as a partial guide.

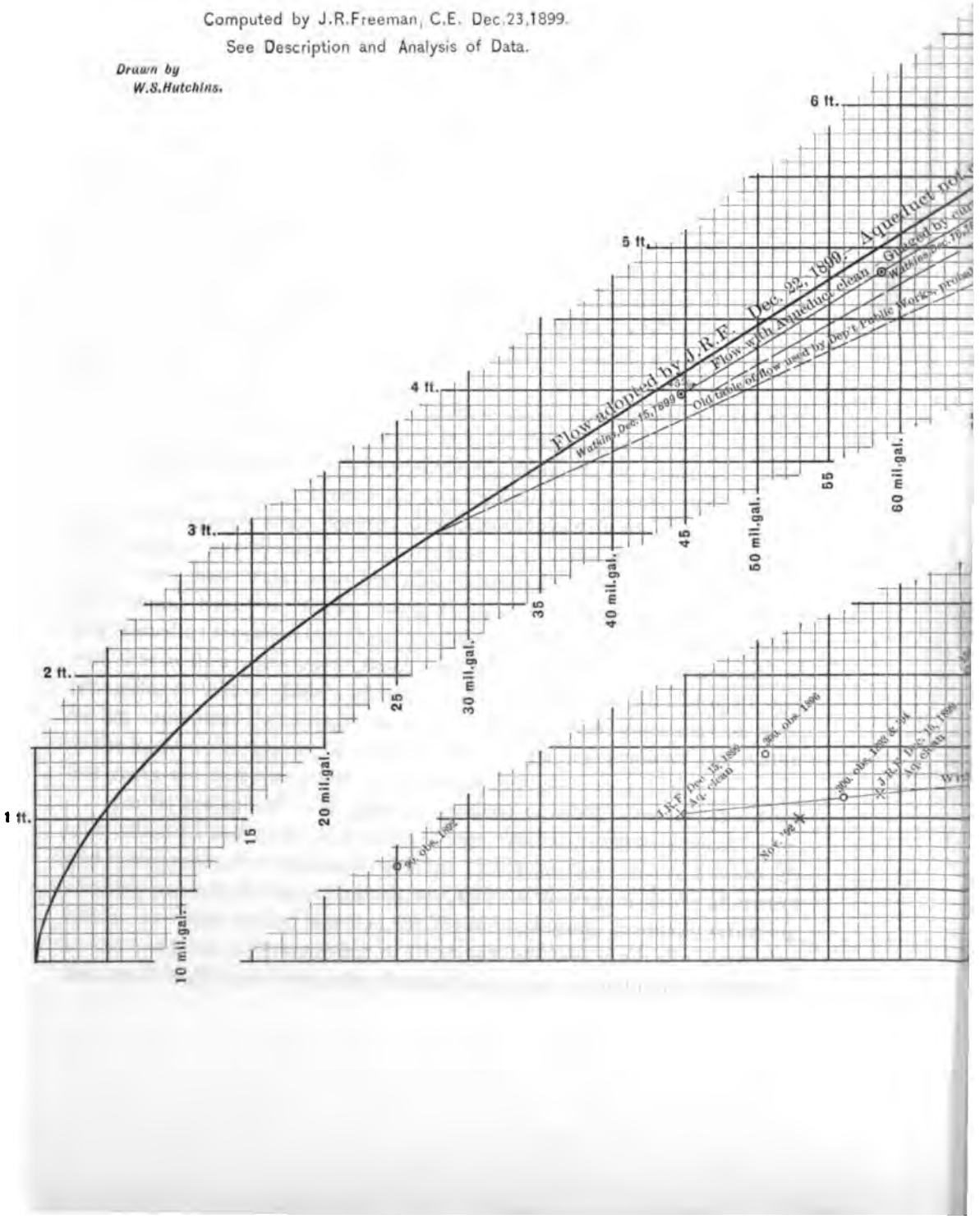
I have been unable to find any definite record or recollection of dates when the Old Aqueduct was cleaned, or of the thoroughness with which this work was done prior to 1890, except the record of the days when the aqueduct was empty, given on page 151, which days are the only times when it could possibly have been cleaned. Doubtless for part of the time since 1892 the condition was so improved by cleaning as to cause it to convey perhaps 5 per cent. more water than the new computation gives, but certainly no more at any time than during our gaugings of December, 1899, for the Old Aqueduct was then in its best condition. On the other hand, it is probable that with a particularly thick coat of slime, or a rank growth of spongilla, or with organic matter an inch thick, such as Mr. Birdsall says he has sometimes found near the upper end, and such collections of slime as the keepers describe as having been seen in the upper mile or two within five years, the quantity would be materially less.

In the further effort to gain the greatest possible accuracy, the dates when the Old Aqueduct was emptied were plotted in chronological order, and it was assumed provisionally that the Old Aqueduct received a thorough cleaning on each of these occasions, and that the flow immediately afterward became the same as that found in the Watkins gauging, gradually returning to the condition corresponding to the mean line and gradually becoming less, until finally just before the cleaning it got into a condition which made

Very likely the resistance of the arch v
filled at this depth (especially when a
settled) exercises great retardation at
depths. No other accurate gaugings
on record for a conduit so nearly fille
soffit of arch.

Fig. 43

DISCHARGE OF OLD CROTON AQUEDUCT
CORRESPONDING TO VARIOUS DEPTHS AS MEASURED AND RECORDED BY KEEPER AT SING SING



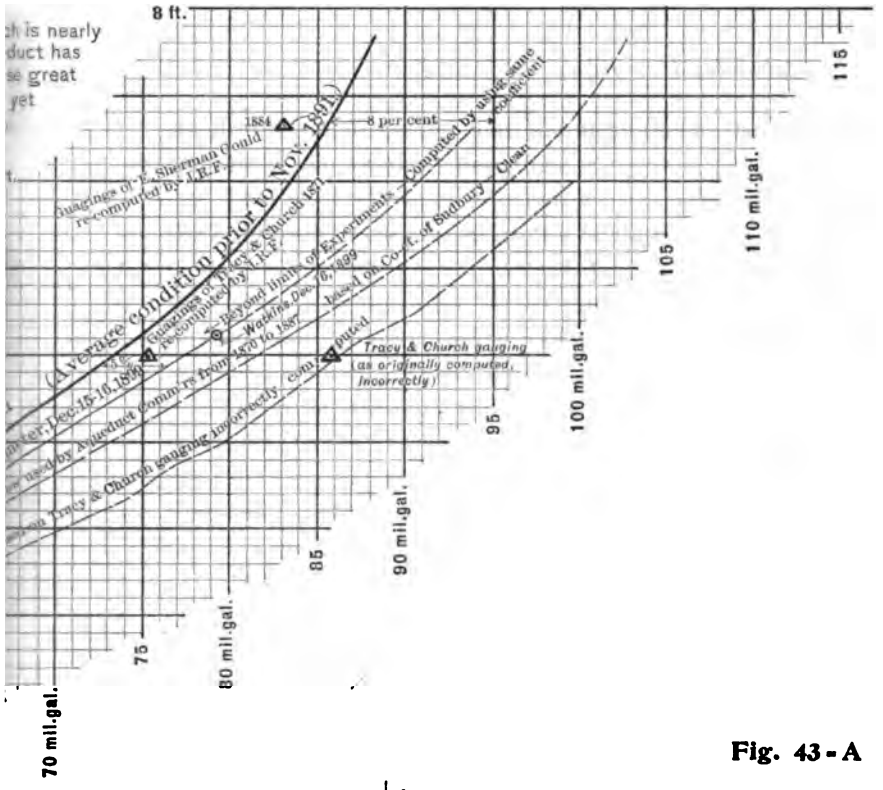
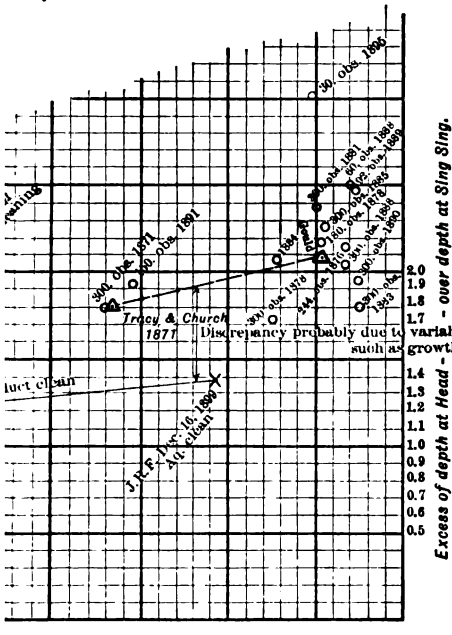


Fig. 43 - A



COMPARISON OF DEPTH AT HEAD OF AQUEDUCT WITH DEPTH AT SING SING

Variation is loss of Head in Upper portion of Old Croton Aqueduct between Sing Sing and Croton Dam.

Probably caused by variable amounts of slime and organic growths adhering to walls of Aqueduct.

Plotted as a means of judging of the comparative cleanliness of interior of Aqueduct at different dates.

(A similar comparison of loss of head in summer and early autumn with loss of head in late winter and early spring shows frequently 6 inches greater loss in late summer and early fall, but this does not occur every year.)

the flow as much smaller than the mean line as the mean line is smaller than the line of the Watkins gauging.

This variation in flow according to cleanliness appears reasonable in the light of the experiments on the Sudbury and Wachusett Aqueducts, which show that one year's collection of slime may lessen the flow 10 per cent., and the recent gauging of the New Croton, which shows that probably because of slime and spongilla or the like, this New Croton Aqueduct has within 7 years suffered 15 per cent. impairment in carrying capacity.

On going through the record month by month and applying such a correction to the average quantity taken from the diagram, the results were not found consistent with the changes in loss of depth between Croton and Sing Sing, and I concluded that such correction was a refinement for which the data did not furnish a satisfactory basis, and that it was just as well to adhere to the average line all the time.

Taking all the various lines of evidence into careful consideration, I am forced to believe that my statement on page 122, that the previous estimates of flow in the Old Croton Aqueduct from 1875 to 1890 averaged too great by 14 million gallons per day, is conservative.

Mr. Gould's series of measurements in 1884 make the delivery 16.2 million gallons less than the Aqueduct Commissioners' average estimate for the same summer and for the same depth.

Determination of Quantity Flowing in New Croton Aqueduct.

I am led to consider it certain that there has been an over-estimate by the Department of Water Supply of about 36 million gallons per day in the quantity of Croton water conveyed to New York City by the New Aqueduct during the last four months of 1899. It is probable that in the spring of 1899 this over-estimate did not exceed 16 million gallons per day, and that the over-estimate has increased gradually from nothing in 1893. The foregoing figures apply to the monthly record computed by Mr. Birdsall, in which he has already taken off a correction of about 4 million gallons per day.

This matter is so important as to require explanation at length.

The number of million gallons per twenty-four hours drawn through the New Aqueduct from day to day for the past nine and a half years has been determined solely by a measurement of the depth flowing in the aqueduct at a point just below the head gates. A table is presented in the Aqueduct Commissioners' Report of 1895, Table No. 14, page 98, which gives the

quantity in million gallons per twenty-four hours conveyed by this aqueduct when the water stands at certain uniform depths. Mr. G. W. Birdsall in preparing the estimates of water consumed, published by the Department of Public Works and the Department of Water Supply, has, of late, deducted about 4 million gallons per day from the figures given by this table, but, in reality, his deduction has been 7 or 8 million gallons per day, because of throwing off all allowance for infiltration and leakage.

This table, No. 14, of the Aqueduct Commissioners' Report of 1895 was designed for use in connection with a depth measurement taken in the aqueduct at a point 490 feet downstream from the head gates (at Station No. 9-65), and includes no allowance for the leakage into the aqueduct along the twenty-five miles of tunnel not under pressure between the Old Croton Dam and the New York City line.

The same table can be used in connection with a depth measured anywhere between Croton and South Yonkers (except near Gould's Swamp Siphon), if only the slope of the water surface be the same as the slope of the bottom of the aqueduct; or, in other words, this table will give the flow for a given depth anywhere in the course of the aqueduct if the depth is precisely uniform for a mile or so upstream and downstream from the point in question. It happens, however, that in the lower reaches of aqueduct, the depth and the slope of water surface are not constant, but, as shown in Sheet No. 40 of plans in Aqueduct Commissioners' Report of 1895, the depths fall off rapidly below Ardsley toward the head of the incline leading to the deep tunnel under pressure. Moreover, the depth in lower part of aqueduct fluctuates to some extent with changes of level in the Central Park Reservoirs, thus complicating the problem, although not making it incapable of accurate solution.

The slope of water surface and the depth are more likely to be uniform near the head of the aqueduct than elsewhere, because of this being beyond the influence of fluctuations in level of the Central Park Reservoirs. Moreover, the quantity ordered sent to the city from day to day must be regulated at the head gates, and the head of the aqueduct is, therefore, the most convenient place for measuring the depth to be used in computing the flow.

We must regard this table (Aq. Com. No. 14), as certainly accurate for the aqueduct in the condition of surface and freedom from obstruction found in 1891 and 1893, although to-day it is not applicable without 14 per cent. correction.

The current meter measurements of 1891 and 1893, made for preparing this table of flowing capacity, were made with the utmost care and confirmed

by frequent repetition between October, 1891, and April, 1893; and the allowance for infiltration was confirmed by independent current meter measurements made from a boat held near the head of the aqueduct and at various points along its course.

The corresponding depths near head gates used in forming Table No. 14, were measured with great care and accuracy by a special observer stationed at the gate-house near the Old Croton Dam; and, although the limit of precision of the current meter measurement was not fully discussed in the report, the agreement of the experiments plotted in diagram on page 97, Aqueduct Commissioner's Report of 1895, give good reason for regarding the table as correct within 1 per cent. for all ordinary depths up to the limit of depth of 7 or 8 feet reached in those experiments, and for the wetted perimeter in the conditions of cleanliness and smoothness existing at that time, when the aqueduct had been in use about two years. I am informed by Mr. Alfred Craven, Acting Chief Engineer, that the aqueduct was probably fairly clean at the time of these gaugings, and that there was probably no special sweeping or scraping of the interior surface precedent to the gaugings. Indeed, a study of the few dates on which it has been practicable to put the aqueduct out of service, shows that no regular cleaning was possible. I infer that it was assumed that because gaugings made after the aqueduct had been in use from $1\frac{1}{2}$ to $2\frac{3}{4}$ years showed no noteworthy change, these would fairly represent the flow in the aqueduct under all ordinary conditions.

Although the depth of water now maintained in the New Croton Aqueduct is greater than in any of these earlier gaugings, recent experiments on the Sudbury Aqueduct indicate that with a corresponding increase of depth, so that half the arch surface is now submerged, there is no very material change in coefficient of flow when aqueduct is clean on account of the greater depth.

Error in Setting Depth Gauge at Head of New Aqueduct.

There is now found to be required a correction for gauge error, which lessens the estimate of daily draft 4 to 10 million gallons per day, this being a part of the 39 million discrepancy found by Mr. Watkins.

I visited the gate-house of the Old and New Aqueducts at Old Croton Dam on October 13, 1899, and had the keeper and his principal assistant instruct me in the methods followed in measuring the various depths which they record, and found that by the method of measuring depth in New Aqueduct by a float gauge, set July 24, 1895, a constant error of at

least 2½ inches (or possibly 3½ inches) had been inadvertently introduced; this leads to an error in the estimate of consumption amounting to from 4 to 10 million gallons per day. I found, too, that this important gauge has got into a condition of extremely sluggish motion, due apparently to the clogging of its piezometer pipe (perhaps by organic growths or deposits similar to those which obstruct the upper portion of the aqueduct). I am told that this float gauge was set by the keeper's assistant without consultation with the engineers of the Aqueduct Commission.

I am told that at about the time of the completion of the aqueduct provision was made by Mr. Fteley for a very accurate gauge at this point, but in the rush of other work the float and index were not completed before the New Aqueduct was formally turned over to the Department of Public Works. A lead pipe for measuring the depth (similar to the present pipe shown in the sketch, Fig. 44) was set up, and a rod was used for direct measurement down from a bench-mark to the surface of water. This was used by the special observers of the Engineer Department of Aqueduct Commission while gauging the flow in 1890–2, but the lead pipe is said to have been injured about 1894, so that the present still-box and piezometer pipe were put in to take its place. The design and mechanism of this new gauge appear excellent. It consists of a scale-board rising from the main upper floor, over which slides an index held steady by a weight and moved by a wire over a pulley at the top, which then descends to a cylindrical copper float of 5 inches diameter by 20 inches long. The index is frequently verified by a measurement to water in the sump chamber, but unfortunately it is continually adjusted on a basis about 2½ inches too high.

The erroneous setting of the present gauge began about 4½ years ago and came from assuming that the elevation of the water in the chamber below the screens would be the same as in the aqueduct, thus ignoring the loss of head due the sharp-cornered contraction at the entrance to aqueduct, and neglecting also the loss of head required to overcome the increase of velocity in passing into the aqueduct. In other words, the float gauge was set to read the same elevation as the still water in the sump chamber, which connects with the screen chamber through a 12-inch pipe.

The following sketch will make the whole matter clear (see also section D, of Gate-house drawings in Aq. Com. Rept. 1887, or Plate No. 69, Wegman).

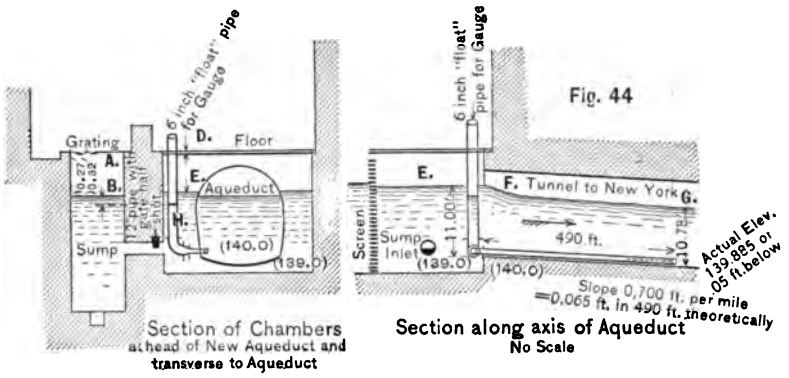


Fig. 44

On October 17, 1899, G. A. Taber, C. E., measuring with an engineer's level and making several repetitions to secure accuracy, found that with a nominal depth in the aqueduct of 10 feet 6 inches, the level of water within the 6-inch vertical pipe, or "still-box," was 0.27 feet lower than water level in the sump at B. About November 1, 1899, a comparison was made under the supervision of C. S. Gowen, Division Engineer Croton Aqueduct Commissioners, while the nominal depth by the float gauge was 11 feet, the screens first having been carefully cleaned to assure an even distribution of the flow. He found the elevation of the water within this 6-inch pipe, 0.32 feet lower than in the sump, and further found by determining the elevation of water in the 6-inch vertical pipe from an accurate bench-mark and comparing this with the known elevation of invert of tunnel at G that the real depth in aqueduct at G was 0.21 feet (2½ inches) less than the depth meanwhile shown by the gauge index, or smaller than the depth found by measuring from the iron floor-grating down to the water level in the sump.

Sluggish action within the float pipe or still-box was also noted by Mr. Taber, and Mr. Gowen, being asked to verify this, found that on filling this pipe with water, it took several hours to subside to its orginal level, most of the time being taken up in the subsidence of the last 2 or 3 inches. The piezometer tube is therefore plainly clogged and ought to be fixed (perhaps spongilla have attached themselves to the piezometer pipe), but changes of water level within the aqueduct are so infrequent that this sluggishness probably introduces no material error into our estimates.

The details of the corrections found necessary at different periods are given in the computation book accompanying this report.

Effect of Infiltration and Leakage upon Delivery of New Croton Aqueduct.

Before preparing Table No. 14 of the Aqueduct Commissioners' Report, 1895, careful measurement had been made of the percolation or infiltration through the rock fissures into the aqueduct, upstream from the gauging station at South Yonkers, and it was concluded that this infiltration amounted to 4 million gallons per twenty-four hours, which amount was deducted from the gauging made at South Yonkers before forming this table of flowing capacity; so that when used in connection with a depth correctly measured near the head gates, this table, No. 14, gives the outflow through the aqueduct from Croton Lake. Experiments reported on page 87 of Aqueduct Commissioners' Report of 1895 gave the outward leakage from 7 miles of the aqueduct under pressure as 228,500 gallons per twenty-four hours. No reason appears why this infiltration and leakage should change materially from year to year, and we may, therefore, consider that the delivery of water into the city when aqueduct is as free of obstruction as in the original experiments, is now, as originally, about 3½ million gallons more than the table gives.

Lessened Delivery of New Croton Aqueduct.

I was led early to question whether the great apparent increase of consumption of water might not, in part, be due to a retardation of the flow in the aqueduct—wherein it is all measured simply by noting the depth—because of the recollection of some experiments upon the retarding influence of organic growths upon the walls of a conduit, in which I assisted Hiram F. Mills, C. E., some twenty-five years ago. Mr. Mills' experiments showed this effect to be far greater than generally supposed. Some data upon the retardation attributed to growths of spongilla and other organisms near the upstream end of the East Jersey Water Company's conduit, and in Rochester's conduit from Hemlock Lake, were also recalled. I was thus led to carefully look up the latest experience in this matter upon the Boston aqueducts, and becoming confirmed in the suspicion that there was some trouble inside the New Croton Aqueduct, earnestly requested the Chief Engineer of the Department of Water Supply, Mr. Birdsall, to have a regauging of the flow in the New Croton Aqueduct, by means of a current meter, for no test of its flowing capacity had been made since 1893.

Mr. Fteley and Mr. Craven very kindly detailed Mr. F. W. Watkins, who made the original current meter measurements, to make these tests. Five separate measurements, made upon November 22 and 23, 1899, and December 5 and 6, 1899, showed the flow to be about 16 per cent., or nearly 40 million gallons per day, less than the flow given by the Aqueduct Commissioners' table of flowing capacity.

TABLE NO. 17.

Gauging of Flow in New Croton Aqueduct.

(By F. W. Watkins, C. E., 1899)

DATE AND DESCRIPTION.	Observed depth at Croton Gate-house, feet.	Corresp. million gallons, flow by Table 14.	Corrected depth at Croton Gate-house.	Corresp. ponding quantity for corrected depth, Table 14, million gallons per 24 hours.	Infiltration as measured in 1890-1892, million gallons.	Nominal rate of flow at Gauging Station.	Observed depth at Gauging Station, South Yonkers.	Area wet section at Gauging Station, sq. ft.	Mean velocity by current meter, feet per sec.	Cubic feet per second gauged.	Uniform depth required to carry this quantity exclusive of infiltration with Aqueduct in normal condition.	Maximum velocity near centre, feet per sec.	Ratio mean velocity to maximum.	Million gallons per 24 hours as gauged.	**Total impairment including error of Croton depth gauge, and allowing for infiltration.**	Impairment after correcting for error of depth of gauge.
Nov. 22, 1899, by series A to I, 103 numbers 20 seconds each	11.00	278.9	10.79	274.2	4	278.23	8.41	105.55	3.545	374.2	9.40	3.87	91.6	241.8	**37.1**	32.4
Nov. 23, 1899, by sub. areas, 1 minute each 105 numbers	11.00	278.9	10.79	274.2	4	278.23	8.45	106.08	3.49	370.2	9.32	....	90.4	239.4	**39.5**	34 8
Nov. 22, 1899, continuous, 102 spaces, 20 seconds each	11.00	278.9	10.79	274.2	4	278.23	8.41	105.55	3.51	370.5	9.32	....	90.7	239.4	**39.5**	34.8
Dec. 5, 1899, by series A to I, 103 numbers, 1 minute each	10.50	267.4	10.29	262.2	4	266.15	8.36	104.89	3.37	353.5	8.95	3.71	90.8	228.4	**39.0**	33.8
Dec. 6, 1899, by series, 103 numbers, 1 minute each	10.50	267.4	10.29	262.2	4	266.15	8.34	104.63	3.37	352.6	8.93	....	90.8	227.9	**38.5**	34.3

The method was precisely the same as that followed in the original gaugings and described in the Aqueduct Commissioners' report of 1895, page 94. The same current meter was used in this series that had been used in the earlier gaugings; its coefficient was redetermined a few days before using and found to agree well with the earlier determinations, as already stated on page 154.

While it is not at present practicable to draw the aqueduct off even for a day for inspection, and while we have no positive information as to the cause of the impairment of its capacity, I am led to believe that this impairment is due to three distinct causes:

1st. Organic growths or other obstructions on bottom and sides of aqueduct interior, more especially in the upper 5 or 10 miles, but also existing to some slight extent in the lower portion.

2d. A serious clogging at the Gould's Swamp Siphon.

3d. The error in measuring the depth, already described.

The keeper's record at the Old Croton Dam shows that the water has not been out of the aqueduct now for four years, and that in all it has been empty less than one month during the past nine and a half years. I am informed that it cannot be emptied at present without interfering with the supply for the high-service pumps near Washington Bridge. On page 59 of the Aqueduct Commissioners' report for 1895 it is stated that its delivery of water to New York City up to that time had been suspended sixty days in all, for the completion of some work at Section 9 and for permitting hydraulic tests.

On some of these sixty days water was in the aqueduct, but was being discharged through the blow-offs, instead of into the Central Park Reservoirs. According to the keeper's record at Croton Dam, the water has been out only as follows:

1890.	Closed, Oct.	1 to 17,	inclusive.....	17	days.
1891.	Closed, Apr.	14 to 18,	"	5	"
1892.	Closed, Dec.	7,	24 hours.....	1	day.
1893.	Closed, Feb.	3,	23.5 "	1	"
1893.	Closed, "	8,	5 "	0.2	"
1893.	Closed, "	14,	20 "	0.8	"
1893.	Closed, "	18,	23 "	1.0	"
1893.	Closed, "	28,	8 "	0.3	"

1895.	Closed, Oct. 15 to 17,	37	hours.....	1.5 days.
1895.	Closed, Nov. 1,	12	"	0.5 day.
	Total days shut off.....			28.3.

This record indicates there has been abundant opportunity for organic growths and for the gathering of such fragments of water-logged, dead leaves as may have passed the coarse screens now in use at the head-gates (these present screens are coarser than good practice sanctions, and I am informed are not the original screens). I am informed that there has been no thorough cleaning of the interior of the aqueduct since it was opened. There has been a little work done in removing obstructions at Gould's Swamp Siphon

Effect of Cleaning Sudbury Aqueduct upon Its Delivery.

On the Sudbury Aqueduct of Boston it has been the rule to give the aqueduct a thorough cleaning twice a year, for, as Mr. Desmond Fitzgerald, for many years in charge of this aqueduct, informs me, " experience on the Cochituate has shown that if deposits of slime and dirt are allowed to accumulate, these in time give place to growths of spongilla, and if these are allowed to accumulate, after a year or two they become very hard, with long finger-like growths extending from them, which are a serious obstruction to the flow of water."

The method followed with the Sudbury until recently (when depths became too large to admit of it), was to run through the aqueduct a machine carried on wheels with soft tires, which was provided with rotary brushes and the whole moved by the pressure of water against an adjustable shield. With the great depths now used, the method of cleaning is to shut off most of the water and send a number of gangs of men through the aqueduct, provided with rubber boots, rattan brooms made especially for the purpose, and reflector lanterns. Mr. Fitzgerald further informs me as to the amount of retardation of flow caused by dirt and slime, that on April 4, 1899, the aqueduct not having been previously cleaned for about one year, an accurate gauging by current meter showed 92.58 per cent. of tabulated capacity, and on May 29, after cleaning, another careful gauging by current meter showed 103.95 per cent. of tabulated capacity. The retardation of flow caused by the condition before cleaning was thus 11 per cent. Mr. Fitz-

gerald states further, "it seems to me safe to say in a general way that the flow can vary 10 per cent. with the condition of the interior, and I can easily conceive that when an aqueduct has been long neglected, this 10 per cent. might be exceeded." Mr. Fitzgerald further states that while in the Sudbury he has found no material difference in the retardation of flow due to growth of slime between the upstream and the downstream portions, he has in former years sometimes noticed in the Old Cochituate Aqueduct a very large growth of spongilla in the upper three miles, while it was free in the lower portion. The Sudbury Aqueduct is less than half the length of the Croton, and it has been so frequently cleaned that spongilla have secured no foothold.

Effect of Slime and Cleaning of Wachusett Aqueduct upon its Delivery.

On the New Wachusett Aqueduct of Boston's supply, which has the same daily carrying capacity as the New Croton, Mr. F. P. Stearns, Chief Engineer of the Metropolitan Water Board, gives me the following information upon the effect of lack of cleanliness in retarding flow. This aqueduct was first used in March, 1898, and its flow measured both by a weir and by a current meter. One year later, March, 1899, with the water at precisely the same depth at the gauging man-hole as in the test of the year before, the flow, as measured by the current meter and independently by the weir, averaged about 10 per cent. less than when first opened and clean.

It was then cleaned by men with scrapers and stiff brooms, and again tested and its flow found very nearly the same for a given depth as at first.

An examination made soon after the second measurement and before cleaning showed a deposit of slime on the sides and bottom of the aqueduct, generally about 1-16 inch in thickness, but, at no place, was there any spongilla or similar growth.

This Wachusett Aqueduct is less than one-third the length of the horseshoe Croton above the gauging station at South Yonkers; and while it is probable that the lower part of a long aqueduct would be less affected than the upstream part, it is the upstream part of the Croton which controls the depth at the head, on which the records of daily flow depend.

Mr. Stearns further informs me regarding the original gaugings of the Sudbury forming the basis of its published coefficients of flow:

"The observations with large flow were made in October and November, 1879. The water was first run through the aqueduct in February, 1878. I think I had the section of the aqueduct, through which the flow was to be

measured, cleaned with brooms before the experiments were made, but am not entirely sure, and have no notes at hand on this point, but the consumption was at that time so small that water flowed only to small depths in the aqueduct until the time of making the experiments."

Condition of Interior Surface of New Croton Aqueduct.

Regarding the condition of the interior surface of the New Croton Aqueduct at the time of the gaugings of 1892, we are not fully informed. Mr. Craven feels confident it was fairly clean, though opportunity for inspection was limited. A study of the uniformity of depth of water on March 2, 1892, through the twenty-five miles of the horse-shoe section, correcting for inclination near the downstream end, shows the coefficient of flow to have been in 1892 substantially the same throughout; and, since careful search was being made for the cause of the 5½ per cent. lessened capacity as compared with the original expectations based on the Sudbury, and gaugings covering a period of 1.3 years showed no noteworthy change, and as the conclusion reached by Mr. Fteley was (see page 101, Aq. Com. Rept. of 1895), that the cause lay in the greater roughness of the surface of the brick interior, we may confidently assume that this possibilty of slime or organic growth existing at that time was not overlooked. A part of this 5 per cent. difference could certainly be accounted for by the extra roughness of brickwork laid in a tunnel, as compared with the Sudbury brickwork laid by daylight in an open cut, and thus leave little to be charged to slime. Wherefore, it would appear that no amount of slime sufficient to affect the flow materially had gathered in the New Croton Aqueduct in these first 2½ years.

The Assistant Engineer (Mr. Horace Ropes) under whose immediate supervision the two-mile tunnel forming a part of the new 300 million gallon aqueduct for the Boston Metropolitan District was built in 1896–7, and who later had charge also of a section of the "cut and cover" aqueduct, tells me that he found it a practical impossibility to get the brick laid to as smooth a surface in the crowded conditions and bad light of the tunnel as by daylight in the open air with fewer brick-layers working from the same line.

It appears at first incredible that a thin coating of organic growth, algæ or slime, which appears scarcely 1-16 inch in average thickness when the water is withdrawn, can produce so great an impairment in flowing capacity as 10 per cent., but the fact is certainly well proved. Possibly when the conduit is filled, the filaments of these algæ or microscopic plants which form

the so-called slime, may reach further out into the current, and the whole may be likened to a microscopic eel-grass. Everyone familiar with natural water courses has noticed how thin and insignificant an obstruction a bed of eel-grass appears when the water is withdrawn, and what a great impediment it is to the current.

When spongilla, a low form of animal life, which investigations by Mr. F. P. Stearns have shown to feed on algæ and micro-organisms, and would, therefore, be expected to thrive best where reservoirs had not had their bottoms stripped of organic matter, are allowed to become well planted and grow, there are developed long, rigid, finger-like projections, which may extend 2 or 3 inches into the current and form a much more prominent and conspicuous obstruction than "slime."

Spongilla appear to thrive best within a few miles of the head of an aqueduct. I cannot learn that any complete study of these matters has been made in connection with the Croton Aqueducts, but there are certain abnormal increases of the friction loss in the current near the head of both the Old Aqueduct and the New Aqueduct, which would be explained in the case of the New Aqueduct during the past summer, if on withdrawing the water there is found either a growth of something like spongilla or a deposit of dead leaves near the head. The progressive increase in backing up of the water near the head of the New Croton Aqueduct during the last summer, from June 7 to September 13, appears to have been fully 6 inches, and possibly a foot, but our data on this are less precise than desirable. Various observations (to be described later) make it appear certain that at the end of the summer of 1899, the total effect of the several obstructions at the upstream end of the New Croton Aqueduct upon the depth, as compared with its condition and depth in April, 1893, was to increase the depth more than 8 inches.

It may be found that on the New Aqueduct, the obstructions liable to occur in the few miles near the head and at the safety gratings of Gould's Swamp Siphon, render a depth measurement at the upstream end of the aqueduct not wholly reliable as a precise measure of the consumption and that some better means of measuring the actual velocity and quantity at frequent intervals, or continuously, is needed.

Possible Retardation at Gould's Swamp Siphon.

This inverted siphon, 14.25 feet in diameter, is only 1,135 feet long, but drops 60 feet or more below the hydraulic grade, and must, therefore, unlike

the horseshoe section, always run with its entire cross section filled. When, as for example, in 1895, the depth at the head was 6.0 feet, the wet area in the horseshoe section of the first twelve miles was 73.01 square feet, and the mean velocity was 2.78 feet per second; meanwhile the area of section of water-way under Gould's Swamp was 159.5 square feet and the velocity only 1.28 feet per second, or less than half that in the horseshoe section above. Considering in the same way the greatest quantity that has yet been run through the New Aqueduct, with the 11 foot depth of October and November, 1899, the actual mean velocity in the horseshoe section was $371 \div 134.5 = 2.76$ feet per second, while under Gould's Swamp it was only 2.32 feet per second, or 85 per cent. as much as if channel were all free. There is thus, without doubt, a tendency of rubbish to lodge in the Siphon and restrict the area to the same or even a less number of square feet than the water area in the Standard Section.

Observations were begun by Mr. Fteley in June, 1899, with a view to discovering if any silting up of the Siphon under the Harlem was going on, and this series of depth measurements was extended up beyond the Gould's Swamp Siphon. These observations were very kindly placed in my hands by Mr. Fteley, although his study of the matter was not then finished, for any light they might give on my inquiry as to possible over-estimate of the daily consumption of water by the city. These observations consisted of measurements of depth of water at various points from Pocantico to One Hundred and Thirty-fifth street, some of them made with much difficulty, if not danger, by descending certain of the shafts used in constructing the aqueduct and measuring to the surface of the water from points of known elevation. Similar observations were again made at the time of Mr. Watkins' current meter gaugings. These depth measurements certainly indicate that there was an obstruction of some kind which at the time of Mr. Watkins' gaugings, November 22 and December 5, 1899, caused an extra loss of head of about 2.3 feet in the Gould's Swamp Siphon, or near its upper end. In going this short distance of 1,169 feet, the normal loss with everything clean should be only about an inch. The possibility of the obstruction of this Siphon is foreshadowed by the Chief Engineer in the Aqueduct Commissioners' report of 1895, at foot of page 87; but the present loss in or near the Siphon, according to the series of measurements above quoted, has increased during the past summer at a rate which it is hard to understand.

Loss of head between Shafts 11 A and 11 B, near head and foot, respectively, of Gould's Swamp Siphon:

June 7, 1899.....	Loss,	1.08 ft.	November 4, 1899....	Loss,	2.19 ft.
July 19, 1899.....	"	1.29 "	November 6, 1899...	"	2.20 "
September 13, 1899...	"	1.51 "	November 23, 1899..	"	2.36 "
October 27, 1899.....	"	2.17 "	December 5, 1899...	"	2.38 "
October 30, 1899.....	"	2.08 "			

On January 16, since writing the foregoing, I am informed that on December 29, Mr. Birdsall had gates shut sufficiently to lower the water to a depth of about 3 feet at head of Gould's Swamp. and sent men in to rake off the safety grating. This was found badly obstructed by leaves and twigs. A small brick dam, perhaps 18 inches high, across bottom of aqueduct built here some years ago was not removed. The loss of head after the above-mentioned cleaning dropped from 2.49 feet to 0.14 feet, but a study of gauge readings still shows excessive friction loss in the upper nine miles of the New Aqueduct, the corrected depth now being 0.86 feet greater at the piezometer 490 feet below the head of aqueduct than at Pocantico.

Probable Obstructions or Foulness near Head of New Croton Aqueduct.

Other observations in the same series when carefully analyzed, allowing for the surface declivity or slope being greater than the slope of the invert, indicate that upstream from the Gould's Siphon there must be independent cause for a backing up of the water 6 inches or a foot near the head-gates, and it looks as though through a combination of circumstances the depth at Pocantico is backed up from the Gould's Siphon obstruction to about the same extent that depth near the head is backed up by obstructions in the aqueduct within, say, five miles of the head; so that the agreement of the Croton gauge with the Pocantico gauge no longer furnished a complete check.

A similar analysis of the depths and surface slopes, or extra inclination below Gould's Swamp and Ardsley, indicates that this part of the aqueduct is in substantially its original condition, or with an impairment of carrying capacity not exceeding 4 per cent. Much more time and many more observations and gaugings, together with an inspection of the interior, are needed to fully establish these matters, and I have set forth at this length the indications from incomplete data, only to emphasize the importance of closer investigation, which I have had no opportunity to make, and because of the great importance of the actual rate of increase in the city's demand for water, whose measurement rests on the correct gauging of the flow in this aqueduct.

A study of the profiles given in Sheet No. 40 of the Aqueduct Commissioners' report of 1895 proves that, coefficient being the same, the flow can

be computed accurately from the depths given at any two points two or three miles apart, located above the gate-house at South Yonkers, by taking the mean of the depths at the two points and looking out the quantity in Aq. Com. Table No. 14 and correcting this proportionally to the square root of the observed slope over the standard slope or fall between the two points. The correctness of this rule for moderate distances was proved in the case of the Sudbury Aqueduct (see p. 94, Boston Additional Water Supply, 1892), and I find it to hold good for the upper section of the Old Croton when clean. The converse must be equally true, and it would be of great interest and value in tracing out the location and extent of obstructions to start upstream from the gauging station at South Yonkers with the line of computed depths for the quantity found by the current meter, with coefficient based on the aqueduct being clean or in same condition as when the table was made, and compare this with the observed profile—thus obtaining the change in coefficient from point to point.

Reviewing the whole of our present data leaves little doubt of the substantial accuracy of Mr. Watkins' gauging, and I have no doubt that with the aqueduct thoroughly cleaned and the gauge corrected, the flow would return to the figures shown by this Table No. 14 of Aq. Com. Rep., '95.

Conclusions Regarding Quantity Carried by New Aqueduct.

The reading of the depth gauge at Croton has first been corrected for the error already described, and we have then looked out the quantity in Aq. Com. Table No. 14 corresponding to the average daily depth, and have assumed that no correction for impairment of carrying capacity of the New Aqueduct was needed for the years 1890 to 1893, inclusive. From the profile of depths it seems clear that the correction discovered by Mr. Watkins' gauging of November 3 may be applied from September 1 to the end of 1899, and that the correction was materially less earlier in the season. By the profile for June 7 I infer that the impairment was then materially less, and in the absence of complete data I have assumed the impairment to have so progressed as to require the following correction to the quantity by Table No. 14, the recorded depths being first corrected for gauge error:

1899—Sept. to Dec., inclusive.....	Deduct	38	million	gallons	per	day.
1899—June to Aug., inclusive.....	"	30	"	"	"	"
1899—Jan. to May, inclusive.....	"	20	"	"	"	"
1898	"	16	"	"	"	"

1897	Deduct	12	million	gallons	per	day.
1896	"	9	"	"	"	"
1895	"	6	"	"	"	"
1894	"	3	"	"	"	"
1893	"	0	"	"	"	"

Conclusions Regarding Observed Yield of Croton Watershed.

In accordance with the foregoing data I have recomputed the flow, day by day and year by year, from the beginning of the records in 1868 up to the end of the year 1899 in a volume which accompanies this report; and have carefully compared this computation, month by month, with the original computation sheets of the Aqueduct Commissioners' Assistant Engineer, which Mr. Fteley very kindly permitted me to examine. The noteworthy differences between the two sets of computations have been carefully reviewed and good explanation found in most cases. With the thousands of separate measurements of depth found in so long a record, averages largely remove the effect of occasional lack of precision of measurement, *and at most, the error in the mean flow for a month or a year of our new Croton record is less than the ordinary error of measurements of a water supply by plunger displacement.*

I believe that while the record of monthly run-off, as it stands after all this adjustment and recomputation, is not perfect, it is not exceeded in accuracy by any record of stream flow in existence for so large a watershed. Of American watersheds that have been accurately gauged for a long period I know of none (except possibly the Sudbury) where the record is more reliable.

The difference between the old estimate of run-off and the new is, as a whole, about 10 per cent. less for the new, varying somewhat from month to month, as aqueduct flow or waste over dam predominates; and after having considered the matter carefully, I venture the opinion that the times are very few when the present estimate of the mean outflow from Croton Lake for any month is so much as 5 per cent. in error, and I believe it probable that the error in the mean run-off for any year rarely, if ever, exceeds 3 per cent.

Whatever uncertainty there is, comes mainly from lack of precision in measurement of depth wasting over dam and from the uncertainty as to the condition of interior surface of the two aqueducts, or obstructions therein, and this, I think, is as great or greater during the past few years than at any other time.

REQUIREMENTS FOR ACCURATE MEASUREMENT OF CROTON YIELD IN FUTURE.

1st. Better means for the accurate measurement of depth should be provided at each gate-house on both the Old Aqueduct and the New Aqueduct.

2d. At stated frequent periods, say once a month, a current meter gauging of the flow in each aqueduct should be made for the verification or correction of the flow computed from observation of the depth. The current meter itself should be re-rated once in three months, or two meters be used, one as a check on the other.

3d. An entirely new set of record books, designed on modern principles, should be begun, and the same care and skill and accuracy given to keeping up a record of daily water consumption and river flow that is given to keeping up the daily pay-roll.

4th. A new rain-gauge of modern form should be set at the Croton Dam Station, and should preferably have no trees or buildings within 100 feet, and be on open, level ground, well away from the steep slope of the hillside.

At least two additional rain-gauges in the northeastern and southeastern corners of the watershed should be established, and the records of all the rain-gauges averaged instead of using the Boyd's Corner gauge alone, in figuring the per cent. of yield.

5th. The daily outflow from each storage reservoir should be measured by noting the area of gate opening or other efficient means, so that the actual draft taken out of storage may be known independently of the volume which disappears from the reservoir under the combined effect of lowering of surface and evaporation.

TABLE NO. 18.

Total Storage Capacity of Croton Reservoirs.

NAME OF RESERVOIR.	Total Visible Capacity up to Present Level of Spillway Crest, Million Gallons.		Authority for Capacity Stated.
	In 1899.	In 1902.	
Artificial Reservoirs.			
Boyd's	2,727	.....	Table of capacity each foot in depth, loaned for copying by G. W. Birdsall.
Middle Branch	4,605	.....	Table of capacity each foot in depth, loaned for copying by G. W. Birdsall.
East Branch Sodom (415)	4,883	.....	Aq. Com. Report, 1895, p. 70, Table 6.
Bog Brook (415)	4,145	.....	Aq. Com. Report, 1895, p. 71, Table 7.
Titicus	7,167	.....	Aq. Com. Report, 1895, p. 73, Table 7½.
West Branch Carmel	10,070	.....	Table of capacity at each foot, loaned for copying by A. Fteley.
Amawalk	7,678	.....	Table of capacity at each foot, loaned for copying by G. W. Birdsall.
Total artificial reservoirs	40,675	40,675	To the last gallon of visible storage. Not including Old Croton Lake.
Controlled Ponds.			
Mahopac	575	.....	Per report of Isaac Newton, Chief Engineer Croton Aqueduct, January 30, 1882, quoted by Wegman, water supply of New York, p. 189. (There are some discrepancies in figures quoted for these ponds in different reports.) Capacities used by Department of Water Supply are slightly different, viz.: Mahopac, 515; Kirk, 500; Gilead, 400; Glenecida, 250; Barrett's, 180; White, 250; total, 2,095. White Pond formerly 100, enlarged in 1898 to 200 million gallons capacity. See D. W. S. Report, 1898. (The total capacity of these, per D. W. S. Report, 1838, p. 4, is 1,800 million gallons.)
Kirk	565	.....	
Glenecida	165	.....	
Gilead	380	.....	
Barrett's	170	.....	
White	200	.....	
Total controlled ponds	2,055	2,055	
Ponds Specially Drawn.			
Peach	230	.....	These ponds are not owned and their outlets are not controlled by the City. Right to draw temporarily was purchased by City for great drought of 1881, and the amounts of water here quoted are understood to have been obtained or available. Wegman's quotation, p. 189 of report of Isaac Newton, Chief Engineer Croton Aqueduct, January 30, 1882. "Some of these were also drawn in 1870." These appear not to have been drawn in great drought of 1891. They are added for quantity available should water famine occur in 1901, but not included in estimate of permanent supply after 1902.
Waccabuc	200	.....	
Cross	110	.....	
China	105	.....	
Pine	75	.....	
Long	60	.....	
Tonetta	50	.....	
Haines	25	.....	
Total special ponds	855	.....	

NAME OF RESERVOIR.	Total Visible Capacity up to Present Level of Spillway Crest, Million Gallons.		Authority for Capacity Stated.
	In 1899.	In 1902.	
Old Croton Lake after 1902 between grades 166.21 and 165.0.	160	160	To utilize Old Aqueduct to 80 million gallons capacity per 24 hours requires that water stand 9.33 feet deep below head gates by many past records, which (see 1880, 1881 and 1876) also show 1.0 foot head required to overcome loss through gates, screens and entrance, bringing pond elevation to about 165, thus affording only 1.2 feet available depth below stone depth.
Old Croton Lake temporarily additional	500	.....	Prior to 1902 can be drawn down 7 feet through New Aqueduct.
Total storage 1899.	44,245	.....	By drawing everything yet built to the utmost gallon, exclusive of the invisible storage of ground water.
Additional in New Croton Lake above Muscoot Sanitary Dam.	.....	2,500	Surveys being incomplete, and Mr. Fteley not caring to go on record with approximate figures, I venture the following analysis. This could be drawn off in a great emergency, although constructed solely to avoid drawing off shallow flowage.
Approximate contents (not yet surveyed) between elevations 154 and 196, between old and new dams, and between elevations 166.23 and 196, betwen old and Muscoot	.....	21,200	The capacity of 32,000 million gallons (p. 82, Aq. Com. Report, 1895) is from grade of bottom of New Aqueduct, elevation 140 to grade of top of proposed flashboards, elevation 200, and includes contents of small Muscoot Dam, per Mr. Fteley's statement to J. R. F. Full flow of New Aqueduct requires 12.85 depth at head and probably at least 1.15 feet loss through screens, angles, gates and entrance section, demanding that lake be not drawn below elevation 154 for full supply. Contents between 140 and 154 not being available, deducts probably at least 2,800 million gallons.
Total	.....	66,590	**Total controlled in 1902 without flash boards on dams.**
Increase possible by flashboards 4 feet high to grade 200	.....	2,800	Plan for stone crest of spillway was lowered from elevation 200 to elevation 196 at request of Department of Public Works. This, I estimate, cuts off about from 2,000 to 2,800 million gallons, which can be regained by flashboards 4 feet high.
Total added by New Croton Lake with flashboards to 200 above level of full aqueduct	26,500	.....	
Total storage 1902.	.....	70,245	Starting with 4-foot flashboards on New Croton dam, and all reservoirs entirely full, including ponds not owned, but drawn in 1870, 1880 and 1881. By drawing everything to the last gallon, not including any allowance for invisible storage of ground water in reservoir margins. This 70,245 million gallons does not include Jerome Park Reservoir, capacity 1,900 million gallons, which is properly a distributing reservoir, and cannot be drawn empty without serious disturbance of water pressure in the city.
Added if Jerome Park Reservoir be emptied		1,900	
		72,145	**Total storage in 1902 with Jerome Park.**

A comparison of the actual storage depletion in the great droughts in 1880 and 1891 with the storage capacity that would have been available had every reservoir been full at beginning of droughts, shows that from one cause and another the storage capacity really available was only about 90 per cent. of the nominal amount. There appears to have been no material assistance from ground water storage but data on ground water here are not very definite.

A storage of 70,000 million gallons gives 203 million gallons per square mile of net land surface.

A storage of 65,000 million gallons gives 189 million gallons per square mile of net land surface.

The total controlled in 1902, without flashboards, 66,590 million gallons, gives 193 million gallons per square mile of net land surface.

TABLE NO. 19.

Croton Drainage Area and Evaporating Surface in Square Miles.

(Grouped according to Main Storage Basins.)

PRESENT COMMON NAME OF RESERVOIR OR POND.	Letter by which it is Designated in Aq. Com. Report.	Date When First Filled and Available.	Approx. Area of Watershed that Drains into this Pond.	Aggregate Area of Watershed of Artificial Storage Reservoir.	Area of Water Surface at Level of Overflow.	Aggregate Area of Water Surface in Watershed of this Reservoir.	*Approx. Area of Swamp Surface per Topog. Map. U.S. Geol.	Net Area of Land Surface.	Per Cent. of Water Surface to Net Land Surface.	REMARKS.
White Pond		1899	0.90		0.176					
Black Pond		....	.....		0.038					
Small Pond		....	.....		0.023					
Boyd's Corner Reservoir	E	1873	20.53	21.43	0.436	0.675	0.	20.75	3.25	
Barretts Pond		1870	0.50		0.108					
Lake Gleneida		1870	0.80		0.284					
Pine Pond		1880	2.0		0.141					
China Pond		1380	0.40		0.095					
Long Pond		....	.....		0.086					
2 small ponds		....	.....	19.51	0.030	2.304	0.3	17.21	13.40	
West Branch Res. Below Boyd's.	D	1895	15.81		1.560					
Total above West Branch Reservoir		....	.....	40.94	.....	2.979	....	37.96	7.85	West Br. Res. receives overflow from Boyd's Cor. Res.
2 small ponds		....	.....		0.094					
Middle Branch Reservoir	G	1878	30.51	20.51	0.672	0 766	0.7	19.74	3.88	
Peach Lake		1880	2.0		0.410					
Croton Pond		....	.....		0.075					
7 small ponds		....	.....		0.102					
East Branch Reservoirs. **Bog Brook Reservoir**	I	1891	74.90	76.90	0.640	2.125	5.0	74.78	2.84	Not flowed above Milltown Bridge.
East Branch Reservoirs. **Sodom Reservoir**	I				0.898					

Lake Mamanassee		....	.....		0.182					
Small Pond		....	.....		0.015					
Titicus Reservoir	M	1893	22.80	22.80	1.103	1.300	0.5	21.50	6.04	
Lake Kirk		1870	2.0		0.158					
Lake Mahopac		1870	2.5		0.875					
Secor Lake		....	.....		0.119					
Mud Pond		....	.....		0.040					
Amawalk (Muscoot) Reservoir	A	1897	13.82	18.32	0.940	2.132	0.05	16.19	13.2	
Lake Gilead		1870	0.6		0.190					
Lake Tonetta		1880	0.9		0.080					
Waccabuc Lake		1880	1.2		0.210					
No. and So. Ponds		....	.....		0.181					
Cross Pond		....	.....		0.160					
Lake Mohansic		....	.....		0.160					
Round Pond		....	.....		c.072					
6 small ponds		....	.....		0.125					
Old Croton Reservoir		1842	156.6	159.3	0.760	1.938	0.7	157.4	1.93	Wegman gives area of Old Croton Lake as about 400 acres.
Total in year 1899	....	....	.....	338.8	.....	11.24	6.75	327.6	3.44	
Lake Colabauch		....	.....		0.045					
Above Colabauch Dam		....	.....		1.920					
New Croton Reservoir		1902	178.2	180.9	3.654	6.797	1.7	174.1	3.9	
Total in year 1902	....	....	.....	360.4	.....	16.10	7.75	344.3	4.68	

* These swamp areas are probably not very accurately outlined on Topographical Map of U. S. Geological Survey.

Modification of Natural Flow of Croton River by Draft from Storage Reservoirs.

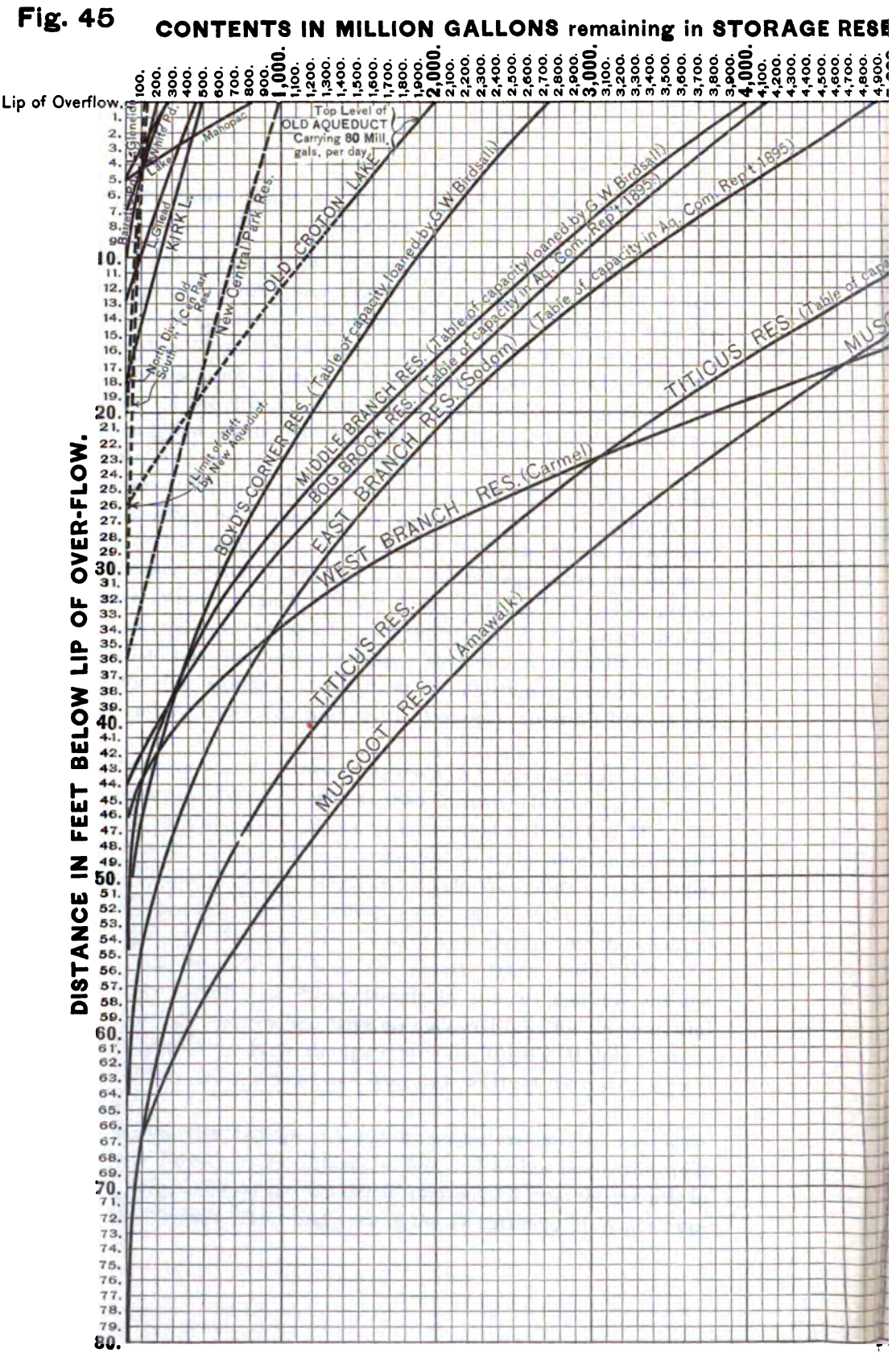
The estimates of yield of the Croton Watershed for each month from 1870 to 1886, given in Tables Nos. 2 and 3 of the Aqueduct Commissioners' report of 1887, and reproduced with extension to the year 1894, as Table No. 13 of the report of 1895, did not contain any allowance whatever for water drawn from storage prior to 1891, and therefore do not properly represent the natural flow of the stream. This I find by personal examination of the original computation sheets, which I was permitted to make through the courtesy of Mr. A. Fteley, Chief Engineer.

For the years 1891 to 1894, inclusive, the observed flow, before insertion in the table published in Commissioners' report for 1895, was corrected for the depletion and refilling of the East Branch and Bog Brook Reservoirs only; no allowance whatever was made for the storage water drawn from Boyd's Corner, Middle Branch, Croton Lake and the controlled ponds. This was discovered not long ago by Mr. Fteley, and for the years 1891 to 1899 the flow had recently been recomputed (but not yet officially printed), by allowing for the storage drawn from all existing reservoirs, except Croton Lake and the controlled lakes, Kirk, Mahopac, Gilead, Gleneida and White.

It was therefore found necessary to make a new computation of this effect of storage upon yield from 1868 to 1899, inclusive, beginning first with the modification of natural run-off due to fluctuation in height of Old Croton Lake, and, from the best records available, to make allowance for the draft and replenishing of the natural ponds, Peach, Waccabuc, Cross, China, Pine, Long, Haines and Tonetta, whose water was bought in the great droughts of 1870 and 1880 and 1881. I have also allowed for the changes from month to month in the levels of the natural lakes, Kirk, Mahopac, Gilead, Gleneida, Barretts and White, bought by the city and now utilized as a part of its storage system; and have also allowed fully for the draft and replenishing of each of the large storage reservoirs, Boyd's, Middle Branch, East Branch, Titicus, West Branch and Amawalk, as these were successively filled and used.

For computing these storage volumes drawn and replenished, the elevation of the water surface or its distance in feet and inches below the spillway crest, has been taken for the last day of each month from the records in the office of Mr. Birdsall, Chief Engineer of the Department of Water Sup-

Fig. 45


CONTENTS IN MILLION GALLONS remaining in STORAGE RESE
DISTANCE IN FEET BELOW LIP OF OVER-FLOW.
Lip of Overflow.
Top Level of OLD AQUEDUCT Carrying 80 Mill. gals. per day.
OLD CROTON LAKE
BOYD'S CORNER RES. (Table of capacity loaned by G.W. Birdsall.)
MIDDLE BRANCH RES. (Table of capacity in Aq. Com. Rep't 1895.)
BOG BROOK RES. (Table of capacity in Aq. Com. Rep't 1895.)
EAST BRANCH RES. (Sodom) (Table of capacity in Aq. Com. Rep't 1895.)
WEST BRANCH RES. (Carmel)
TITICUS RES.
MUSCOOT RES. (Amawalk)
Limit of draft by New Aqueduct
New Central Park Res.
North Div. Old
South " Cent Park Res.
Mahopac
Limit Rd.
Lake
Gilead
KIRKLE
Gilead
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300.
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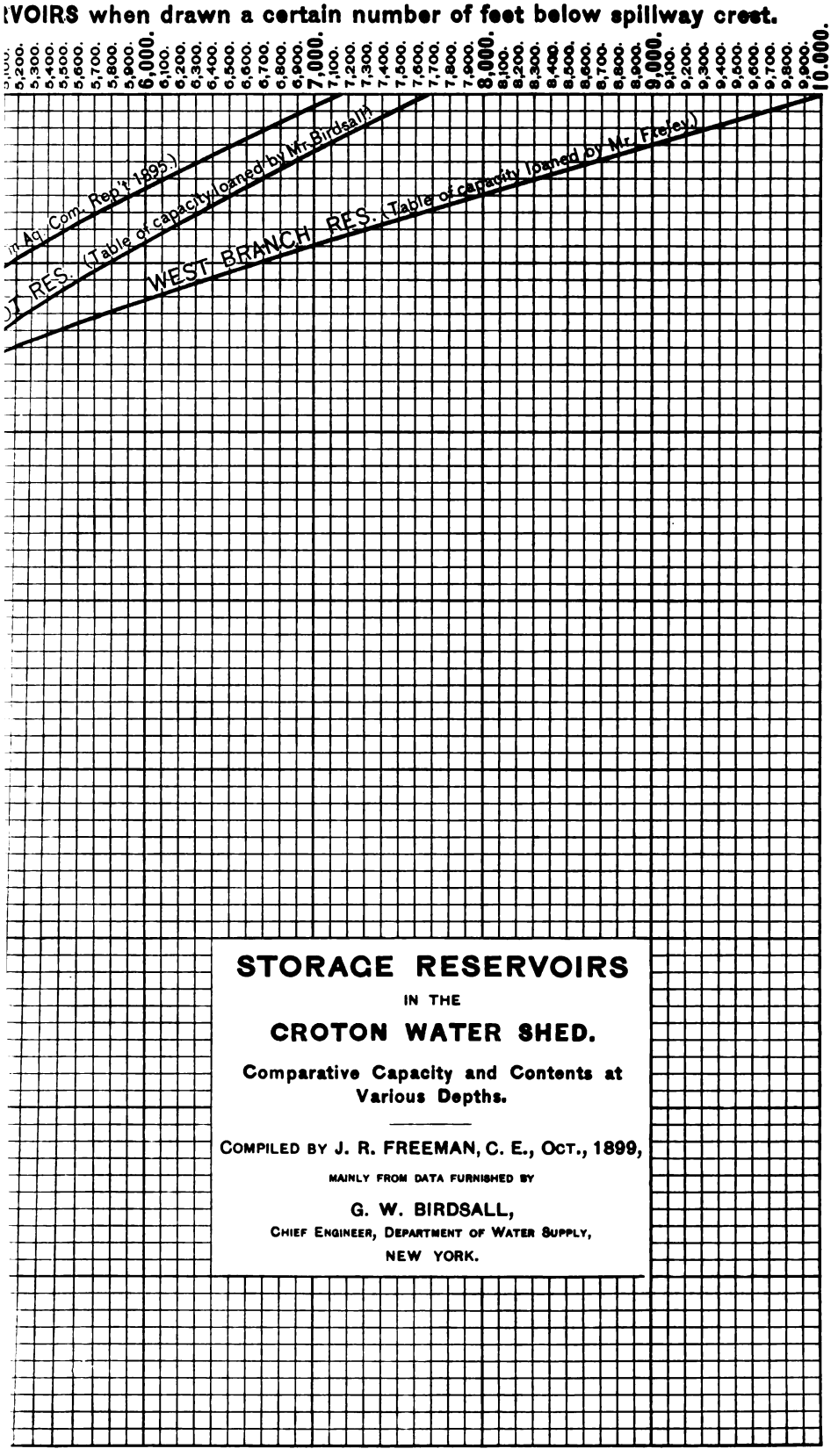
tVOIRS when drawn a certain number of feet below spillway crest.



STORAGE RESERVOIRS

IN THE

CROTON WATER SHED.

Comparative Capacity and Contents at Various Depths.

COMPILED BY J. R. FREEMAN, C. E., OCT., 1899,

MAINLY FROM DATA FURNISHED BY

G. W. BIRDSALL,

CHIEF ENGINEER, DEPARTMENT OF WATER SUPPLY,

NEW YORK.

ply, and applied to the table of storage contents of each reservoir derived from surveys, used by the Department. These tables of storage volumes of each reservoir were converted for convenience of use into a large scale diagram, of which the following reduced copy is of interest for showing the comparative capacity and depth of the different storage reservoirs.

Addition to Visible Storage by Ground Water Storage.

In this computation of amount of water drawn from storage, no allowance has been made for the storage of ground water in the gravelly margins of any of these reservoirs, and the best data that I am able to obtain justifies this course. From carefully studying the rate at which these reservoirs were actually drawn in the great droughts of 1891 and 1895, I can find no indication of material aid from ground water storage properly so called. Mr. G. W. Birdsall, Chief Engineer, Department Water Supply, states that from his knowledge of the Croton Basin and his experience in managing its reservoirs, he believes that any increase of flow from this source must be very small, and that over most of the Croton basins, impervious hard-pan lies within 18 inches of the surface of the ground. Mr. J. J. R. Croes, who supervised the construction of the Boyd's Corner Reservoir, and was on the ground most of the time for five years, tells me that he believes there is very little ground water storage on that basin.

The experienced engineers of Boston's Water Supply, Mr. F. P. Stearns and Mr. Desmond Fitzgerald, while recognizing that ground water storage formed a noteworthy addition to the capacity of the Nashua and Sudbury Reservoirs, in view of the indefiniteness which surrounds the subject, considered it prudent to not include a definite allowance for ground water storage in their estimates of available yield, but let it go as reserve against the possibility of future drought more severe than any yet recorded (see also Fitzgerald, Trans. Am. Soc. C. E., 1892, vol. 2, p. 306).

I have followed a similar course in these computations, and whatever ground water storage may exist is treated, in the present estimate of Croton yield, only as a part of the prudent reserve toward the possibility of the occurrence of some longer period of low rainfall in the future than has occurred in the past thirty-two years. We will not dismiss the subject of ground water storage without some further comment.

There is without doubt some augmentation of the visible storage drawn from these reservoirs, by ground water stored in the saturated sands and

gravels of the reservoir margins, which slowly drains into the reservoir as its level is lowered; but the distinction must be clearly kept in mind between this true ground water storage and the inflow of ground water percolating slowly down the hillside toward the bottom of the valley irrespective of whether a reservoir has been built there or not. Failure to keep this distinction in mind has sometimes led to over-estimates of the effect of ground water storage, and it must be clearly understood that although a considerable stream may come into the bottom of a reservoir for a few weeks while it is drawn to the lowest point, that the total augmentation in per cent. of the entire volume which it can hold may be very small.

Obviously, the per cent. of increased storage that will be so obtained will vary greatly at different reservoir sites. In a broad gravelly plain, like those around some of the Brooklyn Reservoirs, it will be extremely large; while in a hilly, rocky country, with shallow soil, or where the reservoir lies in a "hard pan" formation or "puddling gravel" or in what geologists call the unmodified glacial drift or bowlder clay, it will be extremely small.

We have no data upon the surroundings of the various reservoirs in the Croton Watershed from which this available volume of porous gravel can be estimated, and the opinions of two well-informed engineers familiar with this territory are given above to the effect that this is small.

The only complete measurements on record in this country, so far as known to the writer, of the per cent. of gain or addition to the visible storage by this invisible ground water storage, are those made by Mr. Desmond Fitzgerald upon Basin No. 4 of the Sudbury (see Trans. Am. Soc. C. E., 1892, vol. 2, p. 305), which showed that in that reservoir of 162 acres, rather high in the watershed, in the unmodified sand and gravel and clays of glacial drift, when slowly drawn down 12 feet in two and one-half months, *after allowing for the influx of ground water down the sides of the valley*, as subsequently measured by holding the reservoir level, and also allowing carefully for the actual evaporation at that time and place by an evaporometer regularly observed, it was found that this invisible storage added 8 per cent. to the visible supply.

From such recollections as I have of this Sudbury site in comparison with what I have been able to see of the Croton Reservoir sites, I am inclined to believe that the ground water storage on the average of all the Croton Reservoirs would be less than that found on this one of the Sudbury basins.

To get more light on this matter I carefully studied the records of daily height and of quantity drawn from each Croton storage reservoir day by day in the drought of 1891 (correcting meanwhile for rainfall on and evaporation from the reservoir surfaces), to see if the outflow as measured through the sluices was the same as that measured by the product of the loss in depth and the surveyed area of pond. Unfortunately, at several of the reservoirs the measurement of the delivery ordered from day to day by the Chief Engineer is not made at the sluice, but is measured by opening the gate long enough to drop the water level to a point which, according to the table of reservoir capacities, will yield the volume ordered. Thus, at Boyd's and Middle Branch the ordered volume discharged agrees almost exactly with the volume measured by loss of depth, while at East Branch Reservoir, where it is fairly well established that the outflow was measured by the tables of sluice discharge, understood to have been prepared by the engineers of the Aqueduct Commission, there is a surplus yield averaging at times about 37 per cent. of the visible storage depletion, much the greater part of which appears when the reservoir is nearly empty.

The conditions which exist at East Branch Reservoir are not found at the other Croton Reservoirs. At the head of East Branch Reservoir is a very long and very large gravelly swamp* through which the East Branch river flows with very slight declivity, and the watershed is of 75 square miles area; so the natural slow drainage from this large swamp and the natural yield of a watershed of this size may largely account for this 37 per cent., almost apart from any true ground water storage.

At the other Croton Reservoirs in 1891 there were intermissions in the period of drawing, during which outlet gates were tightly closed. The water rose very slowly during these intermissions.

After all, *the practical question is:*

Can we get an outflow measured through the sluices, or measured into the aqueduct, which will be greater than the volume estimated from the surveyed area and depth of these reservoirs?

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*In the report upon the geology of the Ten Mile River dam sites on page 582, will be found the interesting statement that this swamp is probably the bed of a broad, ancient river which thousands of years ago flowed from the Ten Mile Basin down past Pawling, and filled a much larger channel than the present East Branch of the Croton.

The answer is:

The records of aqueduct delivery during the severest droughts of past years indicate that *the total amount actually obtained from storage was no more than the visible surveyed capacity* and probably was 10 per cent. less (see Depletion Diagram, Fig. 3-B, p. 29, for Nov., 1891, and for 1895).

Correction for Effect of Increased Evaporation from Watershed Due to Substituting Water Surface for Land Surface in the Construction of New Reservoirs.

When the New Croton Lake is filled, eleven square miles of water surface will have been added in the Croton Watershed since 1873, and ten square miles of this added since the drought of 1881.

The evaporation from an exposed water surface is so much greater than from the average land surface that it is the custom of many experienced engineers to deduct the water surface entirely in estimating the area of a watershed. The more accurate experiments of recent years have shown that the loss is not so great as this, and the extended experiments of Mr. Desmond Fitzgerald upon evaporation enable us to estimate the effect with tolerable accuracy.

The evaporation from a water surface being mainly a question of temperature of the air and of its motion, averages very nearly the same from year to year, while varying widely from month to month according to the temperature.

The evaporation from a land surface depends on the wetness of that surface or of the dampness of the earth near the surface as much as upon the temperature, and therefore varies much with variation in rain and wind from year to year.

Our best measure of the evaporation from the land surface is the difference between the rainfall and the run-off of the stream, but this is complicated by the many months required for much of the ground water to find its way from where it falls by slowly percolating through the interstices of the earth toward the bottom of the valley; and the individual gallon of water that sinks during a shower into the square yard of sandy soil high on a side hill may be five years in reaching the stream, while another portion of the same rainfall reaches it in a month through a shorter or more porous course; so to get accurate results, a long period of observations of rainfall and flow must be taken.

Speaking broadly, the annual evaporation from a water surface under conditions fairly comparable with the Croton Reservoirs is 39 inches in depth per year, while from the land surface in an average year the Croton records indicate the evaporation to be 24.9 inches; or, in the average of the four dry years, 1880, 1881, 1891 and 1895, 24.5 inches.

Since it is in dry years and in the dry months that the effect of this loss will be most severely felt, I have made the following computation of the probable average loss in a dry year, month by month. This loss, due to substituting water surface for land, as each successive reservoir site was flooded, though small, is appreciable.

TABLE

Influence of Evaporation from New Reservoirs

Effect of Substituting one Square Mile of Water Surface for one Square

Month.	Run off from Water Surface.									Run off From Net Receipts From Land			
	Mean monthly evaporation in inches, from a water surface, per Fitzgerald Expt's. See transc. A.S.C.E., 1892, p. 275.	Net gain or loss in inches on water surface, being excess or deficiency of the observed rainfall on Croton watershed for the given month and year, over mean evaporation from a water surface per column 2.					Equivalent million gallons per month derived in each of the years from the water surfaces given below.				Natural run off of Net Months			
		Depth in Inches.									Million Gallons per Month from the Net Land Surfaces Given Below.			
		1880.	1881.	1891.	1895.	Average of 1880, 1881, 1891, 1895.	1880. 6.12 Sq. M. Col. 3 x 106.4	1881. 6.12 Sq. M. Col. 4 x 106.4	1891. 7.66 Sq. M. Col. 5 x 133.1	1895. 10.32 Sq. M. Col. 6 x 179.4	1880. Total flow per Table, less Col. 8.	1881. Total flow per Table, less Col. 9.	1891. Total flow per Table, less Col. 10.	1895. Total flow per Table, less Col. 11.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
January....	0.96	+2.68	+3.77	+8.03	+3.45	+4.50	+285	+401	+1069	+619	14,290	4,260	37,080	17,630
February...	1.05	+2.06	+4.20	+4.67	+0.86	+2.95	+219	+447	+ 622	+154	16,240	26,610	31,150	5,600
March.....	1.70	+2.68	+4.45	+1.72	+0.44	+2.32	+285	+474	+ 229	+ 79	16,180	35,570	25 680	20,020
April	2.97	+0.60	—1 59	+0 07	+1.72	+0.20	+64	—169	+ 9	+309	11,420	10,140	15,100	21,810
May	4.46	—3.39	—0.84	—2.84	—2.51	—2.40	—361	— 89	— 378	—450	5.400	7,260	5,180	7,560
June	5.54	—4.15	—0.46	—3.36	—2.99	—2.74	—442	— 49	— 447	—536	2,130	8,900	3,810	2,490
July.	5.98	—0.23	—3.80	—1.89	—0.98	—1.73	— 24	—405	— 252	—176	1,540	2,650	1,740	2,950
August	5. 50	—1.35	—2. 79	—0.32	—1.56	—1.51	—144	—297	— 43	—280	980	750	1,900	1,760
September .	4.12	—1.70	—3.34	—2.10	—3.13	—2.57	—181	—356	— 279	—562	1,340	60	1.870	1,870
October....	3.16	—0.33	—0.34	—0.88	+0.49	—0.27	— 35	— 36	— 117	+ 88	960	2,490	1,470	1,820
November..	2.25	+c.07	+2.98	+1.20	+1.98	+1.56	+ 8	+317	+ 160	+355	2,820	2,650	3.450	3,960
December..	1.51	+1.c9	+4.93	+3.94	+2.96	+3.23	+116	+525	+ 524	+531	2,120	11,260	8,720	6,640
Total..	39.20	—1.97	+7.17	+8.24	+0.73	+3.54	—210	+763	+1097	+131	75,420	112,600	137,150	94,190

The rainfall used in the above is a mean of Boyd's, Croton and Middle Branch, giving weight to each, as follows : This differs slightly from the final adjustment of rainfall records, which include the Carmel gauge.

No. 20.

upon yield of Croton Watershed.

Mile of Land Surface in certain Representative Years of Scant Yield.

Land Surface. Surface and Subsoil.					Water Surface vs. Lands.		Practical Effect in Different Periods.							
Land Surface for the Years and Given Below.					Net Gain or Loss from Substituting 1 Sq. M. of Water Surface for 1 Sq. M. of Land Surface, due evaporation.		Net Gain or Loss in Million Gallons per day upon Yield of Croton Watershed, due to Substitution of Water Surface for Land Surface, to the Extent given Below.							
Equivalent Inches in Depth over Net Land Surface. Quantities for 1 inch in Depth: Mil. Gals. In 1880, 1=332.7 Sq. Mls. =5,782 In 1891, =331.1 Sq. Mls. =5,754 In 1895, =328.5 Sq. Mls. =5,709														
1880.	1881.	1891.	1895.	Average of 1880, 1881, 1891, 1895.	In Inches, Depth.	In Million Gallons per day.	11.10 Sq. M. Water substituted between 1868 and 1902.	10.67 Sq. M. Water substituted between 1874 and 1902.	10.00 Sq. M. Water substituted between 1878 and 1902.	9.10 Sq. M. Water substituted between 1891 and 1902.	8.46 Sq. M. Water substituted between 1892 and 1902.	7.36 Sq. M. Water substituted between 1895 and 1902.	5.80 Sq. M. Water substituted between 1896 and 1902.	4.86 Sq. M. Water substituted between 1897 and 1902.
16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
2.47	0.74	6.45	3.10	3.19	+1.31	+0.73	+ 8.1	+ 7.8	+ 7.3	+ 6.6	+ 6.2	+ 5.4	+ 4.2	+3.5
2.81	4.60	5.41	0.98	3.45	—0.50	—0.31	— 3.4	— 3.3	— 3.1	— 2.8	— 2.6	— 2.3	— 1.8	—1.5
2.80	6.15	4.47	3.50	4.23	—1.91	—1.07	—11.9	—11.4	—10.7	— 9.7	— 9.1	— 7.9	— 6.2	—5.2
1.97	1.75	2.62	3.83	2.54	—2.34	—1.36	—15.1	—14.5	—13.6	—12.4	—11.5	—10.0	— 7.9	—6.6
0.93	1.26	0.90	1.32	1.10	—3.50	—1.95	—21.8	—20.9	—19.6	—17.8	—16.6	—14.4	—11.4	—9.5
0.37	1.54	0.66	0.44	0.75	—3.49	—2.00	—22.2	—21.3	—20.0	—18.2	—16.9	—14.7	—11.6	—9.7
0.27	0.46	0.30	0.52	0.39	—2.12	—1.19	—13.2	—12.7	—11.9	—10.8	—10.1	— 8.8	— 6.9	—5.8
0.17	0.13	0.33	0.31	0.24	—1.75	—0.98	—10.9	—10.5	— 9.8	— 8.9	— 8.3	— 7.2	— 5.7	—4.8
0.23	0.01	0.32	0.33	0.22	—2.79	—1.62	—18.0	—17.3	—16.2	—14.7	—13.7	—11.9	— 9.4	—7.9
0.17	0.43	0.26	0.32	0.30	—0.57	—0.32	— 3.6	— 3.4	— 3.2	— 2.9	— 2.7	— 2.4	— 1.9	—1.6
0.49	0.46	0.60	0.69	0.56	+1.00	+0.58	+ 6.4	+ 6.2	+ 5.8	+ 5.3	+ 4.9	+ 4.3	+ 3.4	+2.8
0.37	1.95	1.52	1.16	1.25	+1.98	+1.11	+12.3	+11.8	+11.1	+10.1	+ 9.4	+ 8.2	+ 6.4	+5.4
13.05	19.48	23.84	16.50	18.22	Loss, 14.68 inches per year.	Loss, 0.70 inches per year.	Used from Jan., 1868, to Feb., 1874.	Used from Mar., 1874, to July, 1878.	Used from Aug., 1878, to Mar., 1891.	Used from April, 1891, to Feb., 1892.	Used from Mar., 1892, to Mar., 1895.	Used from April, 1895, to Feb., 1896.	Used from Mar., 1896, to Mar., 1897.	Used from April, 1897, to Dec., 1902.

Boyd, 3; Croton, 2; Middle Branch, 5, in total of 10.

Evaporation Loss from Reservoirs.

Having introduced the values found by Table 20 in column 4 of the book of computations of storage depletion, see Table No. 24 for sample page, no further correction need be made for loss in storage volume by evaporation from the natural ponds, controlled lakes and storage reservoirs that were in existence at time of the given measurement.

To be more specific, take the case of September, 1881, the total water surface with reservoir full was 6.1 square miles.

The observed measured outflow from Croton Lake was 85.4 million gallons per day for 30 days..	2,562	million	gallons.
Within these 30 days, the lowering of the reservoirs and ponds corresponds to a release from visible storage of	2,855	"	"
But part of this storage went out by evaporation; probably 4.12 inches in depth over 6.1 square miles was lost by evaporation, while 0.78 inch was gained by rain falling directly into reservoirs, making a net loss of 3.34 inches, equivalent to	356	"	"
Leaving to flow down into the aqueduct from storage only	2,499	"	"
Whence we might say that *had there been no part* of the 6.1 square miles of *water surface* and no storage reservoirs, the natural run-off of the stream would have been.....	63	"	"

But that is not what we want. We want, as a step toward further computations, to know the natural outflow from this watershed containing this 6.1 square miles of water surface. And this we get by disregarding the evaporation and subtracting the total depletion of visible storage from the observed outflow, 2,562 — 2,855 = minus 293 million gallons.

There are two months found where the natural flow thus derived comes out a minus quantity.

This being a minus quantity does not mean that no water would actually have run out past Croton Dam, but means that the watershed as a whole sustains a loss during the month which must be made good sometime later.

At the next step in our estimates, that of computing the amount of

storage needed for a greater rate of draft by the aqueducts, the same 6.1 square miles of original water surface will in future suffer the same evaporation that it did while the earlier measurement of outflow was being made; and thus by the method followed, having first corrected for the effect of the 10 square miles of water surface added between 1881 and 1902 in causing evaporation loss, we introduce no error into the computation by ignoring evaporation from the original water surface.

No allowance has been made for the variation in evaporation due to the change in the area of water surface as the reservoir lowers.

To exhibit clearly the successive steps in our method of estimating the actual flow, day by day and month by month, for the past thirty-two years, and the changes that the construction of new reservoirs will introduce, and the future yield possible from the same rainfall as in the past, we present the following sample pages from the record during the drought of 1891.

A volume of about 500 similar pages, containing these computations for the past thirty-two years in detail, will be handed you to be submitted to such further examination as may be desired, or to be filed with the records of the Water Department for future reference.

TABLE 21.

(Sample page from Computation Book of 384 pages—one for each month of the year.)

Computation of Average Daily Flow of Croton River for the Month of November, 1891.

Day of Month.	Depth of Water above or below Lip of Old Croton Dam. 5 A.M.	Depth of Water above or below Lip of Old Croton Dam. 6 P.M.	Waste over Old Croton Dam in Million Gallons per 24 hours.	Old Aqueduct. (Depth given is commonly the mean of 3 daily observations.) Depth at about 600 feet below Head Gates.	Depth at Sing Sing Ventilator.	Excess Depth at Head above Sing Sing.	Depth at Tarrytown.	Computed Delivery in Million Gallons per 24 hours, per Sing Sing Depth.	Hour of Changing Gates to Aqueduct.	New Aqueduct. Mean of 3 daily observations. Depth at Screen Chamber or Sump.	Depth at Station 9+65 by Stilox and Pipe.	Mean Depth Used.	Depth at Tarrytown, 9 A.M.	Delivery in Million Gallons per 24 hours, per Table No. 13, Aq. Com. Report.	Total Out-flow from Croton Lake in Million Gallons per 24 hours.	Rain or Melted Snow at Croton Dam.
	Inches.	Inches.		Feet.	Feet.	Feet.	Feet.			Feet. *A	Feet. †B	Feet.	Feet.			
1.....	—73	—74	0	0	0	0	0	0	.....	6.33	6.21	6.21	6.08	137.9	.....	....
2.....	—75	—75	0	0	0	0	0	0	.....	6.33	6.21	6.21	6.08	137.9	.....	....
3.....	—76	—77	0	0	0	0	0	0	.....	6.33	6.21	6.21	6.08	137.9	.....	....
4.....	—80	—81	0	0	0	0	0	0	.....	6.33	6.21	6.21	6.08	137.9	.....	....
5.....	—81	—81	0	0	0	0	0	0	.....	6.33	6.21	6.21	6.08	137.9	.....	....
6.....	—83	—84	0	0	0	0	0	0	.....	6.33	6.20	6.20	6.08	137.6	.....	....
7.....	—85	—87	0	0	0	0	0	0	.....	5.83	6.20	6.20	6.08	137.6	.....	....
8.....	—89	—91	0	0	0	0	0	0	.....	5.83	6.20	6.20	5.42	137.6	.....	....
9.....	—95	—97	0	0	0	0	0	0	.....	5.75	5.56	5.56	5.42	116.8	.....	....
10.....	—101	—103	0	0	0	0	0	0	.....	5.07	5.51	5.51	5.42	116.2	.....	....
11.....	—105	—106	0	0	0	0	0	0	.....	5.62	5.49	5.49	5.42	115.6	.....	.28
12.....	—104	—103	0	0	0	0	0	0	.....	5.64	5.51	5.51	5.42	116.2	.....	.24
13.....	—103	—104	0	0	0	0	0	0	.....	5.33	5.21	5.21	5.42	107.1	.....	....
14.....	—105	—106	0	0	0	0	0	0	.....	5.33	5.21	5.21	5.42	107.1	.....	....
15.....	—107	—107	0	0	0	0	0	0	.....	5.00	5.21	5.21	5.00	107.1	.....	....
16.....	—108	—109	0	0	0	0	0	0	.....	5.00	4.87	4.87	5.00	96.8	.....	....
17.....	—109½	—108	0	0	0	0	0	0	.....	5.00	4.87	4.87	5.00	96.8	.....	.34
18.....	—93	—91	0	1.50	0	1.50	0	0	6 P.M.	5.04	4.89	4.89	5.00	97.5	.....	.80
19.....	—87	—85	0	1.50	1.33	.17	1.37	7.0	.....	5.00	4.91	4.91	5.00	98.1	.....	....

20.....	—85	—82	0	1.50	1.42	.08	1.37	7.8	.....	5.00	4.91	4.91	5.00	98.1	.....	....
21.....	—81	—81	0	1.50	1.42	.08	1.33	7.8	.....	5.00	4.88	4.88	5.00	97.2	.....	....
22.....	—81	—80	0	1.50	1.42	.08	1.33	7.8	.....	5.00	4.88	4.88	5.00	97.2	.....	....
23.....	—80	—78	0	1.50	1.42	.08	1.33	7.8	.....	5.00	4.89	4.89	5.00	97.5	.....	....
24.....	—60	—48	0	2.00	1.42	.58	1.42	7.8	10 A.M.	5.00	4.89	4.89	5.67	97.5	.....	1.36
25.....	—40	—38	0	2.50	1.83	.67	1.67	11.9	.....	6.00	5.86	5.86	5.67	126.9	.....	....
26	—36	—35½	0	2.50	2.08	.42	2.08	14.7	.....	6.00	5.86	5.86	5.67	126.9	.....	....
27.....	—35	—35	0	3.00	2.08	.92	2.08	14.7	10 P.M.	6.00	5.89	5.89	5.67	127.8	.....	.25
28.....	—33½	—33	0	3.00	2.08	.92	2.50	14.7	.....	6.00	5.85	5.85	5.58	126.7	.....	.04
29.....	—34	—34	0	3.00	2.42	.58	2.50	19.2	.....	6.00	5.85	5.85	5.67	126.7	.....	....
30,.....	—39	—42	0	3.00	2.42	.58	2.50	19.2	.....	6.00	5.88	5.88	5.67	127.6	.....	....
Total.	....	....	0	....	....	....	....	140.4	.....	....	....	....	....	3.527.7	.....	3.31
Av. per day for month.	....	....	0	....	....	....	....	4.7	.....	....	....	....	....	117.6	122.3	

Infiltration less leaks and use by Prison	+3.2
Net delivery to Manhattan and Bronx	125.5
Average daily gain or loss in distributing reservoirs	—1.2
Average daily consumption and waste of Croton water in City for month	124.

Former Department Public Works Estimate—Croton Consumption.

Result of Aqueduct Commissioners' computation, Waste, 0.

+ Old Aqueduct, 30)170. / 5.6 + New Aqueduct, 30)3720 / 124 =	129.6
Storage, 30)108 / 3.6	3.6
	126.0

Table No. 13, Aqueduct Commissioners' Report, reads 129.0, instead of 126.0.

* New Aqueduct depth—A, per keepers' record. † New Aqueduct depth—B, per Aqueduct Commissioners' observer

TABLE No. 22.

(Sample page from Computation Records.)

Flow of Croton River—Computation of Effect of Storage.

(In Augmenting or Diminishing mean monthly flow.

For *storage volumes* of the several reservoirs at various depths, see diagram No. 44, plotted from tables loaned by G. W. Birdsall.

The following computation is based upon the distance in feet of *water level* below crest of spillway, observed on *last day of each month* (main reservoirs have gauge reading each day; controlled ponds are observed once each week, and the latest observation of the month is here taken). Distance below spillways transcribed from records in office of G. W. Birdsall. Effect of evaporation and ground water storage not here included. (See remarks on page .) Quantity of water stated in million gallons; depths stated in feet and decimal (depth is observed in even inches).

Reservoir.	July, 1891.				August, 1891.				September, 1891.			
	Distance below Spillway at end of Month.	Total Contents at this depth.	Gain or Loss for Month. †	Effect on Daily River Flow.	Distance below Spillway at end of Month.	Total Contents at this Depth.	Gain or Loss for Month. †	Effect on Daily River Flow.	Distance below Spillway at end of Month.	Total Contents at this Depth.	Gain or Loss for Month. †	Effect on Daily River Flow.
Boyd's	11.33	1,780	720	....	19.75	1,210	570	....	28.83	700	510	....
Middle Branch	10.17	2,690	800	....	18 25	1,800	890	....	25.83	1,100	700	....
East Branch	10.00	3,310	905	....	21.67	1,930	1,380	....	37.92	710	1,220	....
Mahopac L	0.33	755	45	....	0.25	770	—15	....	1.75	510	260	....
Kirk L	15.50	65	365	....	13 50	115	—50	....	12.75	135	—20	....
Gleneida L	0.00	250	0	....	0.00	250	0	....	0.00	250	0	....
Gilead L	0.42	435	5	....	0.33	440	—5	....	1.17	405	35	....
Barrett's Pd	7.00	45	110	....	7.50	40	5	....	7.50	40	0	....
Old Croton L.*	2.92	1,750	200	....	2.42	1,790	—40	....	2.50	1,780	10	....
Total effect of storage	.....	9,580	2,950	+95.2	.....	6.845	2,735	+88.2	.....	4,130	2,715	+90.5
	October, 1891.				November, 1891.				December, 1891.			
Boyd's	46. 58	80	620	....	42 00	190	—110	....	24.67	910	—720	....
Middle Branch	38.83	240	860	....	42.50	110	130	....	31.83	640	—530	....
East Branch	53.75	110	000	....	55.33	80	30	....	36.33	800	—720	....
Mahopac L	3.17	275	235	....	2.96	310	—35	....	2.33	410	—100	....
Kirk L	13.33	120	15	....	16.33	40	80	....	12.00	155	—115	....
Gleneida L	2.58	100	150	....	4.25	25	75	....	3.75	45	—20	....
Gilead L	7.33	170	235	....	10.50	65	105	....	9.50	95	—30	....
Barrett's Pd	7.50	40	0	....	7.17	45	—5	....	6.00	60	—15	....
Old Croton L.*	5.83	1,510	270	....	3.25	1,720	—210	....	—1.17	2,000	—280	....
Total effect of storage	.....	1,145	2,985	+96.3	.....	1,085	60	+2.0	.....	3,615	—2,530	—82.0

* While Old Croton Lake is said to have a total nominal capacity of about 2,000 million gallons, only about 500 million gallons of this is available, therefore the unavailable portion is deducted from the total in the footing of the column of total contents.

† The minus sign denotes loss to the river flow and gain to volume in storage.

TABLE 23.

(Sample page from Computation Book).

Computation of Effect of Storage in Distributing Reservoirs on Average Daily Consumption of Croton Water.

The following computation is based upon the distance in feet of *water level* below crest of spillway, observed on *last day of each month.* Distance below spillway transcribed from records in office of G. W. Birdsall. Effect of evaporation and rainfall not included here. Quantity of water stated in million gallons; depths, in feet and decimal.

Reservoir.	July, 1891.				August, 1891.				September, 1891.			
	Distance Below Spillway on Last Day of Month.	Total Contents at this Depth.	Gain or Loss for Month.	Effect on Average Daily Consumption.	Distance Below Spillway on Last Day of Month.	Total Contents at this Depth.	Gain or Loss for Month.	Effect on Average Daily Consumption.	Distance Below Spillway on Last Day of Month.	Total Contents at this Depth.	Gain or Loss for Month.	Effect on Average Daily Consumption.
Old Central Park, Northern Division	20.25	0	0	....	20.25	0	0	....	20.25	0	0	....
" Southern Division	2.50	101.	—1	....	2.50	101.	0	....	4.83	92.	—9	....
New Central Park, Eastern and Western Divisions	2.25	920.	—10	....	2.33	915.	—5	....	5.00	838.	—77	....
Forty-second Street	6.50	21.	0	....	6.25	21.5	+1	....	10.00	18.5	—3	....
Total	....	....	—11	+0.3	....	....	—4	+0.1	....	....	—89	+2.9
	October, 1891.				November, 1891.				December, 1891.			
Old Central Park, Northern Division	20.25	0	0	....	20.25	0	0	....	20.25	0	0	....
" Southern Division	7.58	81.	—11	....	14.58	54.	—27	....	0.25	110.	+56	....
New Central Park, Eastern and Western Divisions	6.58	790.	—48	....	4.33	855.	+66	....	0.25	980.	+124	....
Forty-second Street	10.83	18.	—1	....	16.83	13.5	—4	....	7.17	20.5	+7	....
Total	....	...	—60	+1.9	....	....	+35	—1.2	....	....	+187	—6.0

A summary of the estimates for each month follows, in tabular form:

TABLE 24.

(Sample page from Computation Records.)

Croton River Average Daily Flow during each Month of 1891.

1st Column. *Record of actual outflow* from old Croton Lake at Water Works Dam as computed from daily observations of depth wasting over dam and depth drawn in aqueducts.

2d Column. Increase in mean daily flow for the month that has come from drawing off artificial storage reservoirs and controlled ponds; also decrease in flow caused by refilling the same.

3d Column. *Natural flow* that would have occurred without storage (without special correction for evorporation from reservoirs and without correction for ground-water storage in reservoir margins—these being temporarily assumed to offset each other).

4th Column. Computed *modification*, by evaporation of the flow found at the given date that would be expected from substitution of water surface for land surface in the reservoirs constructed between the given date and 1902.

5th Column. Computed increase in natural flow due *increased drainage* area (21.6 square miles = 6.4 per cent.) controlled by new dam in 1902.

6th Column. Computed flow resulting from the rainfall of the given year under the changed conditions of 1902.

7th, 8th and 9th Columns. *Storage required* for various rates of consumption.

All flows and corrections estimated in million gallons per 24 hours.

1890. Computed on.....1899, by..... Checked by.....

Month. December is of preceding year.	1 Observed Flow.	2 Increase Due Storage.	3 Natural Flow.	4 Evaporation Loss.	5 Gain Due 6.4% Increased Water Shed.	6 Computed Flow Exclusive of Storage from Same Rainfall After 1902.	7 With Consumption Averaging 250 Million Gallons per 24 Hours.		8 With Consumption Averaging 275 Million Gallons per 24 Hours.		9 With Consumption Averaging 300 Million Gallons per 24 Hours.	
							Surplus or Deficiency This Month.	Total Depletion of Storage.	Surplus or Deficiency This Month.	Total Depletion of Storage.	Surplus or Deficiency this Month.	Total Depletion of Storage.
December	809.5	—4.3	805.2	+11.1	51.5	868	+19,160	.....	+18,380	.....	+17,610	....
January	391.0	—0.8	390.2	+7.3	25.0	423	+5,360	.....	+4,590	.....	+3,810	.....
February	617.1	+0.5	617.6	—3.1	39.5	654	+11,310	.....	+10,610	.....	+9,910	.....

March	843.5	+1.0	844.5	−10.7	54.0	888	+19,780	.....	+19,000	.....	+18,230	.....
April	588.3	+0.3	588.6	−13.6	37.7	613	+10,890	.....	+10,140	.....	+9,390	.....
May	457 8	+3.0	460.8	−19.6	29.5	471	+6,850	.....	+6,080	.....	+5,300	.....
June	269.3	−0.3	269.0	−20.0	17.2	266	+480	.....	−270	270	−1,020	1,020
July	143.8	−17.2	126.6	−11.9	8.1	123	−3,940	3,940	−4,710	4,980	−5,490	6,510
August	105.8	−17.2	88.6	−9.8	5.7	84	−5,150	9,090	−5,930	10,910	−6,700	13,210
September	374.7	+19.3	394 0	−16.2	25.2	403	+4,590	4,500	+3,840	7,070	+3,090	10,120
October	587.8	+8.0	595.8	−3.2	38.1	631	+11,810	(7,310)	+11,030	(3,960)	+10,260	(140)
November	369.7	−0.3	369.4	+5.8	23.6	399	+4,470	.....	+3,720	.....	+2,970	.....
Total	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1891.												
December	290.9	+3.0	293.9	+11.1	18.8	324	+2,290	.....	+1,520	.....	+470	.....
January	1,230.3	+0.4	1,230.7	+7.3	78.8	1,317	+33,080	.....	+32,300	.....	+31,530	.....
February	1,136.3	−1.6	1,134.7	−3.1	72.6	1,204	+26,710	.....	+26,010	.....	+25,310	.....
March	756.4	+79.2	835.6	−10.7	53.5	878	+10,470	.....	+18,700	.....	+17,920	.....
April	440.4	+63.3	503.7	−12.4	32.2	524	+8,220	.....	−7,470	.....	+6,720	.....
May	161.0	−6.0	155.0	−17.8	9.9	147	−3,190	3,190	−3,970	3,970	−4,740	4,740
June	141.6	−29.5	112.1	−18.2	7.2	101	−4,470	7,660	−5,220	9,190	−5,970	10,710
July	143.1	−95.2	47.9	−10.8	3.1	40	−6,510	14,170	−7,280	16,470	−8,060	18,770
August	148.2	−88.2	60.0	−8.9	3.8	55	−6,040	20,210	−6,820	23,290	−7,590	26,360
September	143.5	−90.5	53.0	−14.7	3.4	42	−6,240	26,450	−6,990	30,280	−7.740	34,100
October	140.0	−96.3	43.7	−2.9	2.8	44	−6,390	32,840	−7,160	37,440	−7 940	42,040
November	122.3	−2.0	120 3	+5.3	7.7	133	−3,510	36,350	−4,260	41,700	−5,010	47,050
December	214.9	+82.0	296.9	+10.1	19.0	326	+2,360	33,990	+1,590	40,110	+810	46,240
Total	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

TABLE

Influence of Evaporation from New Reservoirs

Effect of Substituting one Square Mile of Water Surface for one Square

Month.	Run off from Water Surface.										Run off From Net Receipts From Land			
	Mean monthly evaporation in inches, from a water surface, per Fitzgerald Expt's. See transc. A. S. C. E., 1892, p. 275.	Net gain or loss in inches on water surface, being excess or deficiency of the observed rainfall on Croton watershed for the given month and year, over mean evaporation from a water surface per column 2.					Equivalent million gallons per month derived in each of the years from the water surfaces given below.				Natural run off of Net Months			
		Depth in Inches.									Million Gallons per Month from the Net Land Surfaces Given Below.			
		1880.	1881.	1891.	1895.	Average of 1880, 1881, 1891, 1895.	1880. 6.12 Sq. M. Col. 3 x 106.4	1881. 6.12 Sq. M. Col. 4 x 106.4	1891. 7.66 Sq. M. Col. 5 x 133.1	1895. 10.32 Sq. M. Col. 6 x 179.4	1880. Total 332.7 Sq. M. flow per Table, less Col. 8.	1881. Total 352.7 Sq. M. flow per Table, less Col. 9.	1891. Total 331.1 Sq. M. flow per Table, less Col. 10.	1895. Total 328.5 Sq. M. flow, per Table, less Col. 11.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
January....	0.96	+2.68	+3.77	+8.03	+3.45	+4.50	+285	+401	+1069	+619	14,290	4,260	37,080	17,620
February...	1.05	+2.06	+4.20	+4.67	+0.86	+2.95	+219	+447	+ 622	+154	16,240	26,610	31,150	5,630
March.....	1.70	+2.68	+4.45	+1.72	+0.44	+2.32	+285	+474	+ 229	+ 79	16,180	35,570	25 680	20,020
April	2.97	+0.60	−1 59	+0 07	+1.72	+0.20	+64	−169	+ 9	+309	11,420	10,140	15,100	21,810
May	4.46	−3.39	−0.84	−2.84	−2.51	−2.40	−361	− 89	− 378	−450	5.400	7,260	5,180	7,560
June	5.54	−4.15	−0.46	−3.36	−2.99	−2.74	−442	− 49	− 447	−536	2,130	8,900	3,810	2,490
July.	5.98	−0.23	−3.80	−1.89	−0.98	−1.75	− 24	−405	− 252	−176	1,540	2,650	1,740	2,950
August	5. 50	−1.35	−2.79	−0.32	−1.56	−1.51	−144	−297	− 43	−280	980	750	1,900	1,760
September.	4.12	−1.70	−3.34	−2.10	−3.13	−2.57	−181	−336	− 279	−562	1,340	60	1.870	1,870
October....	3.16	−0.33	−0.34	−0.88	+0.49	−0.27	− 35	− 36	− 117	+ 88	960	2,490	1,470	1,820
November..	2.25	+c.07	+2.98	+1.20	+1.98	+1.56	+ 8	+317	+ 160	+355	2,820	2,650	3.450	3,960
December .	1.51	+1.09	+4.93	+3.94	+2.96	+3.23	+116	+525	+ 524	+531	2,120	11,260	8,720	6,640
Total ..	39.20	−1.97	+7.17	+8.24	+0.73	+3.54	−210	+763	+1097	+131	75,420	112,620	137,150	94,190

The rainfall used in the above is a mean of Boyd's, Croton and Middle Branch, giving weight to each, as follows : This differs slightly from the final adjustment of rainfall records, which include the Carmel gauge.

No. 25.

at Old Croton Dam, including Storage Draft.

Freeman, C. E., February, 1900.)

WATER SURFACE EXPOSED TO EVAPORATION.

5.8 sq. m., 1868 to 1873 — 1.73 per cent.
6.2 " 1873 to Oct., 1878—1.83.
6.9 " 1878 to 1891 — 2.03.
8.4 " 1891 to 1893 — 2.48.
9.5 sq. m., 1893 to 1895 — 2.82 per cent.
11.0 " 1895 to 1897 — 3.28.
12.0 " 1897 to 1900 — 3.56.

1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.
102	241	134	490	234	458	124	397	188
777	672	274	725	516	528	855	830	652
547	1,208	1,175	657	724	494	1,027	882	493
987	1,101	513	287	892	356	322	243	476
304	334	148	255	318	148	228	390	225
97	117	103	251	152	89	294	291	117
94	90	84	120	120	89	95	114	85
925	84	82	109	238	85	85	85	85
146	66	60	409	191	85	85	581	86
137	65	175	153	105	84	85	376	110
380	120	755	360	135	98	80	163	117
297	79	327	1,327	320	89	298	216	104
396	346	319	427	327	216	295	378	225

1891.	1892.	1893.	1894.	1895.	1896.	1897.	1898.	1899.
1,230	650	325	287	514	225	240	653	695
1,136	270	558	401	200	454	416	950	503
756	328	1,114	881	586	1,011	535	573	1,326
440	213	619	422	690	511	440	386	723
161	253	1,048	322	227	102	494	852	217
142	214	183	274	165	201	262	340	227
143	160	162	167	165	187	465	207	220
148	156	189	161	171	202	534	387	229
144	172	164	169	179	194	221	226	225
140	150	216	158	164	195	194	253	228
122	271	273	426	158	284	199	367	228
215	285	569	387	165	200	393	584	227
394	261	452	338	282	321	367	479	425

TABLE

NATURAL *Average Daily Flow of Croton River at Old Croton Dam, including*

(Estimate of John R.

Computed from observed flow by deducting the quantity corresponding to lowering of Storage Reservoirs or adding the quantity taken to refill, without allowance for evaporation loss from Storage Reservoirs.

Average for each calendar month in million gallons per 24-hour day.

MONTHS.	1868.	1869.	1870.	1871.	1872.	1873.	1874.
January	332	380	672	98	347	729	1,578
February	150	371	810	416	217	311	532
March	742	945	600	595	292	687	564
April	665	592	785	344	541	1,267	633
May	1,013	484	303	343	209	361	536
June	577	229	141	244	205	85	154
July	186	102	81	117	99	84	237
August	350	58	78	143	276	125	141
September	960	35	30	106	208	88	63
October	503	453	48	338	182	240	136
November	692	364	110	654	463	293	110
December	264	547	101	381	236	568	165
Average for year	536	381	310	314	272	404	405

MONTHS.	1884.	1885.	1886.	1887.	1888.	1889.	1890.
January	404	738	636	545	706	775	390
February	1,935	477	926	976	892	444	618
March	874	345	426	644	832	355	844
April	532	458	834	596	903	460	589
May	321	267	362	270	458	294	461
June	103	68	138	208	242	250	269
July	147	37	87	459	67	272	127
August	199	75	70	613	188	719	89
September	103	25	26	156	611	404	394
October	64	83	32	178	435	324	596
November	199	461	184	169	555	970	369
December	689	408	254	420	932	805	294
Average for year	387	286	324	429	567	506	419

No. 26.

Storage Draft with Watershed and Reservoirs as Existing in the Given Year.

Freeman, February, 1900.)

Drainage area 338.8 square miles.

WATER SURFACE EXPOSED TO EVAPORATION.

5.8 sq. m., 1868 to 1873 — 1.73 per cent.
6.2 " 1873 to Oct. 1878 — 1.83
6.9 " 1878 to 1891 — 2.03
8.4 " 1891 to 1893 — 2.48
9.5 sq. m., 1893 to 1895 — 2.82 per cent.
11.0 " 1895 to 1897 — 3.28
12.0 " 1897 to 1900 — 3.56

1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.
91	241	143	490	235	465	150	491	191
809	672	288	725	516	562	966	906	652
550	1,208	1,237	657	724	525	1,163	885	499
1,004	1,101	513	296	892	377	332	244	479
304	334	146	276	306	158	231	392	226
91	111	103	269	156	56	295	292	115
60	47	40	124	128	49	72	83	66
965	48	44	105	224	27	15	19	28
147	66	48	428	197	39	10	657	10
137	62	206	164	69	30	79	384	93
381	133	821	386	101	94	90	165	145
297	71	327	1,354	277	72	380	219	115
460	339	326	438	317	203	310	392	215

1891.	1892.	1893.	1894.	1895.	1896.	1897.	1898.	1899.
1,231	913	278	292	584	383	371	652	692
1,135	321	687	404	201	850	603	953	571
836	413	1,246	883	645	1,367	647	570	1,331
504	257	620	424	743	556	467	387	724
155	279	1,049	322	225	104	513	856	172
112	209	173	259	65	167	253	318	81
48	106	60	48	89	125	500	85	178
60	183	121	48	48	98	571	526	41
53	91	93	119	44	119	159	143	152
44	42	229	100	61	156	99	189	91
120	320	359	641	144	334	275	450	139
297	273	708	451	231	252	498	649	150
379	284	469	332	257	374	413	479	359

TABLE

Average Daily Flow of Croton River at New Croton Dam, Exclusive of Storage

Estimates of John R. Freeman,

Computed from natural flow at old dam for the given year and month by adding 6.4% for increase in drainage area and deducting an allowance for the evaporation loss caused by substituting water surface for land surface in the New Reservoirs constructed between the given year and 1902.

MONTHS.	1868.	1869.	1870.	1871.	1872.	1873.	1874.
January	362	412	723	113	377	784	1,687
February	156	391	859	439	328	327	562
March	777	993	627	621	299	719	589
April	693	615	821	351	560	1,333	659
May	1,056	493	300	344	201	302	549
June	592	221	127	237	196	68	143
July	184	96	73	112	92	76	240
August	361	50	72	141	283	122	140
September	1,004	19	14	95	204	76	50
October	531	478	48	356	190	251	141
November	743	394	123	702	500	318	123
December	293	595	120	418	263	617	167
Average for year	563	397	322	326	290	421	423

MONTHS.	1884.	1885.	1886.	1887.	1888.	1889.	1890.
January	437	792	652	587	759	832	423
February	1,098	504	983	1,035	946	470	654
March	919	357	443	674	875	367	888
April	552	474	874	620	947	476	613
May	322	264	365	214	478	293	471
June	90	52	127	202	238	246	266
July	145	27	80	477	59	277	123
August	202	70	64	643	190	755	84
September	93	10	11	150	634	414	403
October	65	85	31	186	460	341	631
November	217	497	202	186	597	1,038	399
December	745	445	281	458	1,003	868	324
Average for year	405	297	338	450	597	532	438

NO. 27.

Draft, with Rainfall as in the given year and with Reservoirs as in 1902.

C. E., February, 1900.

Total drainage area 360.4 square miles, of which 16.1 square miles = 4.47% is water surface.
Average for calender month in million gallons per 24-hour day.

1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.
105	264	160	529	258	502	167	530	210
857	712	303	768	546	595	1,025	961	691
574	1,274	1,304	688	760	548	1,226	931	520
1,053	1,157	531	300	936	380	340	246	496
402	335	135	273	306	149	227	398	221
75	96	88	264	146	40	294	291	102
51	37	30	119	124	40	65	83	58
1,016	40	37	102	229	19	6	10	20
139	53	33	439	194	25	—27	682	—5
142	62	216	171	70	29	81	405	96
411	148	880	417	114	106	111	181	160
328	87	360	1,452	305	88	415	244	134
426	353	340	459	330	209	323	410	222

1891.	1892.	1893.	1894.	1895.	1896.	1897.	1898.	1899.
1,317	978	302	317	628	412	400	698	740
1,204	339	728	47	211	902	640	1,913	606
878	430	1,316	930	678	1,448	682	601	1,411
524	262	649	440	781	583	490	406	764
147	280	1,099	326	225	100	537	901	173
101	206	168	258	55	166	260	329	76
40	102	54	41	86	126	526	84	184
55	187	120	43	44	99	602	555	39
42	83	86	113	35	117	162	144	154
44	42	240	104	63	165	103	199	95
133	345	387	687	157	359	296	476	150
326	300	763	490	254	275	536	696	165
396	297	492	347	269	394	436	506	379

TABLE

Computed Natural Flow of Croton

(Estimates of John R.

Total number of million gallons delivered in each month exclusive of storage draft. The figures below are the monthly aggregates of the daily flows in Table 27.

MONTHS.	1868.	1869.	1870.	1871.	1872.	1873.	1874.
January	11,220	12,770	22,410	3,500	11,690	24,300	52,300
February	4,520	10,950	24,050	12,290	9,510	9,160	15,740
March	24,090	30,780	19,440	19,250	9,270	22,290	18,260
April	20,790	18,450	24,630	10,530	16,800	39,990	19,770
May	32,740	15,280	9,300	10,660	6,230	11,220	17,020
June	17,760	6,630	3,810	7,110	5,880	2,040	4,290
July	5,700	2,980	2,260	3,470	2,850	2,360	7,440
August	11,190	1,550	2,230	4,370	8,770	3,780	4,340
September	30,120	570	420	2,850	6,120	2,280	1,500
October	16,460	14,820	1,490	11,040	5,890	7,780	4,370
November	22,290	11,820	3,690	21,060	15,000	9,540	3,690
December	9,080	18,450	3,720	12,960	8,150	19,130	5,800
Total for year	205,960	145,050	117,450	119,090	106,160	153,870	154,520
Average daily flow	563	397	322	326	290	421	423
Average daily flow, per square mile.	1.561	1.103	0.894	0.905	0.805	1.170	1.174
Total six consecutive dry months....	94,840	38,370	13,810	38,990	35,740	27,780	25,630
Daily average, six dry consecutive months, per square mile	1.430	0.582	0.208	0.591	0.539	c.421	0.389

MONTHS.	1884.	1885.	1886.	1887.	1888.	1889.	1890.
January	13,550	24,550	20,210	18,200	23,530	25,790	13,110
February	31,840	14,110	27,520	28,980	27,430	13,160	18,310
March	28,490	11,070	13,730	20,890	27,130	11,380	27,530
April	16,560	14,220	26,220	18,600	28,410	14,280	18,390
May	9,980	8,180	11,320	6,630	14,820	9,080	14,600
June	2,700	1,560	3,810	6,060	7,140	7,380	7,980
July	4,500	840	2,480	14,790	1,830	8,590	3,810
August	6,260	2,170	1,980	19,930	5,890	23,400	2,600
September	2,790	300	330	4,500	19,020	12,420	12,090
October	2,010	2,640	960	5,770	14,260	10,570	19,560
November	6,510	14,910	6,060	5,580	17,910	31,140	11,970
December	23,090	13,790	8,710	14,200	31,090	26,910	10,040
Total for year	148,280	108,340	123,330	164,130	218,460	194,100	159,990
Average daily flow	405	297	338	450	597	532	438
Average daily flow, per square mile	1.124	0.824	0.938	1.248	1.656	1.476	1.215
Total six consecutive dry months....	24,770	15,690	15,620	56,630	62,950	63,870	58,010
Daily average, six consecutive dry months, per square mile..	0.376	0.237	0.237	0.859	0.949	0.979	0.880

No. 28.

River at Site of New Croton Dam.

Freeman, C. E., February, 1900.)

With rainfall as in the given year and with evaporating surface of Reservoirs as in 1902.

1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.
3,260	8,180	4,960	16,400	8,000	15,560	5,180	16,430	6,510
24,000	20,650	8 480	21,500	15,290	17,260	28,700	26,910	19,350
17,790	39,490	40,430	21,330	23,560	16,990	38,010	28,860	16,120
31,590	34,710	15,930	9,000	28,080	11,400	10,200	7,380	14,880
12,460	10,380	4,180	8,460	9,490	4,620	7,040	12,340	6,850
2,250	2,280	2,640	7,920	4,380	1,200	8,820	8,730	3,060
1,580	1,150	930	3,690	3,840	1,240	2,010	2,570	1,800
31,530	1,240	1,150	3,160	7,100	590	190	310	620
4,170	1,590	990	13,170	5,820	750	—810	20,460	—150
4,400	1,920	6,700	5,300	2,170	900	2,510	12,560	2,980
12,330	4,440	26,400	12,510	3,420	3,180	3,330	5,430	4,800
10,170	2,700	11,160	45,010	9,450	2,730	17,860	7,560	4,150
155,530	129,330	123,950	167,450	120,600	76,420	118,040	149,540	80,970
426	353	340	459	330	209	323	410	222
1.182	0.980	0.942	1.273	0.916	0.579	0.896	1.138	0.616
56,260	13,040	16,590	41,700	26,730	7,860	16,050	48,890	13,110
0.853	0.197	0.250	0.629	0.405	0.119	0.243	0.737	0.199

1891.	1892.	1893.	1894.	1895.	1896.	1897.	1898.	1899.
40,830	30,320	9,360	9,830	19,470	12,770	12,400	21,640	22,940
33,710	9,830	20,380	11,960	5,910	26,160	17,920	28,370	16,970
27,200	13,330	40,800	28,830	21,020	44,890	21,140	18,630	43,750
15,720	7,860	19,470	13,200	23,430	17,490	14,700	12,180	22,920
4,560	8,680	34,070	10,110	6,980	3,100	16,650	27,930	5,360
3,030	6,180	5,040	7,740	1,650	4,980	7,800	9,870	2,280
1,240	3,160	1,670	1,270	2,670	3,910	16,300	2,600	5,700
1,700	5,800	3,720	1,330	1,360	3,070	18,060	17,210	1,210
1,260	2,490	2,580	3,390	1,050	3,510	4,860	4,320	4,620
1,360	1,300	7,440	3,220	1,950	5,110	3,190	6,170	2,950
3,990	10,350	11,610	20,610	4,710	10,770	8,880	14,280	4,500
10,110	9,300	23,650	15,190	7,870	8,520	16,610	21,580	5,120
144,710	108,600	179,790	126,680	98,070	144,280	159,110	184,780	138,320
396	297	492	347	269	394	436	506	379
1.100	0.823	1.365	0.963	0.746	1.094	1.210	1.405	1.051
12,580	27,610	32,060	27,060	13,390	23,680	59,690	54,450	21,260
0.151	0.416	0.486	0.408	0.203	0.357	0.905	0.826	0.322

TABLE

Average Daily Flow per Square Mile of Total Drainage Area at New Croton Dam,

(Estimates of John R. Freeman,

Computed by dividing quantities in Table No. 27, by 360.4 square miles.
In thousand gallons per 24-hour day.

MONTHS.	1868.	1869.	1870.	1871.	1872.	1873.	1874.
January	1,004	1,143	2,006	314	1,045	2,175	4,682
February	433	1,085	2,384	1,216	910	908	1,560
March	2,156	2,755	1,740	1,723	830	1,995	1,634
April	1,923	1,706	2,278	974	1,554	3,699	1,829
May	2,931	1,368	832	955	558	1,005	1,523
June	1,643	613	352	658	544	189	397
July	511	256	203	311	255	211	666
August	1,002	139	200	391	785	339	388
September	2,786	53	39	264	566	211	139
October	1,473	1,326	133	988	527	606	391
November	2,062	1,093	341	1,948	1,387	882	341
December	813	1,651	333	1,160	730	1,712	519
Average for year	1,561	1,103	894	905	805	1,170	1,174
Average for driest 6 consecutive months of the given year	1,430	582	208	591	539	421	389

MONTHS.	1884.	1885.	1886.	1887.	1888.	1889.	1890.
January	1,213	2,198	1,809	1,629	2,106	2,309	1,174
February	3,047	1,398	2,828	2,872	2,625	1,304	1,815
March	2,550	991	1,229	1,870	2,428	1,016	2,464
April	1,532	1,315	2,425	1,720	2,628	1,321	1,701
May	894	733	1,013	594	1,326	813	1,307
June	250	144	352	561	660	683	738
July	402	75	222	1,324	163	769	341
August	561	194	178	1,784	527	2,095	233
September	258	28	31	416	1,739	1,149	1,118
October	180	236	86	516	1,276	946	1,751
November	602	1,379	560	516	1,657	2,880	1,107
December	2,067	1,235	780	1,271	2,783	2,409	899
Average or year	1,124	824	938	1,248	1,656	1,476	1,215
Average for driest 6 consecutive months of the given year	376	237	237	859	949	979	880

No. 29.

exclusive of storage draft, with Rainfall as in the given year and Reservoirs as in 1902.

C. E., February, 1900.)

Total drainage area, 360.4 square miles. Land, 344.3 square miles, 95.53 per cent.
Water, 16.1 square miles = 4.47 per cent. of total area or 4.68 per cent. of net land.

1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.
291	733	444	1,468	716	1,393	463	1,470	583
2,378	1,975	841	2,131	1,515	1,051	2,844	2,667	1,916
1,593	3,535	3,619	1,909	2,109	1,520	3,402	2,583	1,443
2,922	3,211	1,473	832	2,597	1,054	943	683	1,376
1,116	930	375	758	849	413	635	1,104	613
208	266	244	732	405	111	816	867	283
142	103	83	330	344	111	180	230	161
2,819	111	103	283	635	53	17	28	56
386	147	92	1,218	538	69	—75	1,893	—14
394	172	599	475	194	81	225	1,124	265
1,140	411	2,442	1,157	316	294	308	502	444
910	241	999	4,030	846	244	1,152	677	372
1,182	980	942	1,273	916	579	896	1,138	616
853	197	250	629	405	119	243	737	199

1891.	1892.	1893.	1894.	1895.	1896.	1897.	1898.	1899.
3,654	2,714	838	880	1,743	1,143	1,110	1,937	2,053
3,343	940	2,020	1,185	586	2,503	1,776	2,811	1,681
2,436	1,193	3,651	2,581	1,881	4,018	1,892	1,667	3,915
1,434	727	1,801	1,221	2,167	1,618	1,360	1,127	2,120
408	777	3,050	905	624	277	1,490	2,500	480
280	572	466	716	158	461	722	913	211
111	283	150	114	238	350	1,460	833	511
152	519	333	119	122	275	1,671	1,540	108
116	230	239	313	98	325	450	400	427
122	116	666	289	175	458	286	552	264
769	957	1,074	1,906	436	996	821	1,321	416
905	832	2,117	1,360	705	763	1,487	1,932	458
1,100	823	1,365	963	746	1,094	1,210	1,405	1,051
191	416	486	408	203	357	905	826	322

TABLE

Average Daily Consumption of Croton Water in New York for
(Estimates of John R. Freeman,

As re-computed from data furnished by G. W. Birdsall, Chief Engineer, Department Water Supply, A. Fteley, Chief Engineer Croton Aqueduct Commission, and Daily Records of Keepers at Croton Dam, Sing Sing, Tarrytown and Central Park.

MONTHS.	1868.	1869.	1870.	1871.	1872.	1873.	1874.
January	68	70	76	71	77	83	81
February	71	70	75	72	78	81	84
March	68	75	73	70	77	81	83
April	64	71	69	72	73	84	82
May	67	75	68	73	76	79	82
June	67	73	72	72	75	82	83
July	65	72	72	74	77	81	85
August	72	64	67	73	74	77	85
September	65	56	67	72	71	80	84
October	71	57	64	74	75	83	84
November	64	58	69	73	76	76	84
December	67	63	69	76	74	82	89
Average for year	68	67	70	72	75	81	84

MONTHS.	1884.	1885.	1886.	1887.	1888.	1889.	1890.
January	87	87	88	89	91	84	87
February	83	89	88	86	90	88	86
March	84	84	78	84	87	85	87
April	82	83	80	82	83	87	87
May	86	82	84	85	82	88	88
June	86	84	87	85	88	89	87
July	85	84	88	88	86	84	98
August	84	83	84	86	87	88	108
September	82	85	86	88	85	80	123
October	81	80	84	83	85	86	132
November	85	82	83	84	85	86	125
December	88	85	87	84	86	87	133
Average for year	84	84	85	85	86	87	103

No. 30.

each Month for the past 32 years in Million Gallons per 24 hours.
C. E., February, 1900.)

Tables of Aqueduct flow for given depths being corrected in accordance with gauging of F. W. Watkins, C. E., November and December, 1899, and a re-computation of Tracy & Church gaugings of Old Aqueduct in 1871, and E. Sherman Gould gauging of 1884.

1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.
89	87	78	84	93	82	88	85	88
92	87	79	85	88	85	86	84	88
86	84	83	83	84	83	82	83	86
83	83	81	84	81	84	82	84	82
79	84	81	81	80	87	82	84	84
87	87	84	83	84	84	85	87	83
86	87	85	85	84	82	85	86	85
85	85	82	84	80	81	86	86	84
87	74	78	86	81	84	84	84	83
83	70	63	86	85	83	79	85	84
84	00	81	83	84	81	77	83	87
86	88	83	84	83	86	82	86	83
86	81	80	84	84	83	83	85	85

1891.	1892.	1893.	1894.	1895.	1896.	1897.	1898.	1899.
137	149	168	159	158	175	206	204	218
135	155	177	161	174	177	207	206	233
142	160	163	158	167	173	202	202	226
139	148	172	157	158	161	193	197	218
144	149	153	162	157	169	190	195	218
146	155	161	170	166	190	193	205	228
146	162	165	171	166	192	197	209	224
151	162	165	163	173	198	194	213	232
149	161	167	164	183	198	201	223	227
145	153	166	162	166	197	196	217	230
124	151	163	167	160	196	190	210	230
141	154	162	161	167	204	193	207	224
142	155	165	163	166	186	197	207	226

PROBABLE SAFE FUTURE DRAFT FROM CROTON WATERSHED.

Having, as previously described, and as illustrated in the sample pages from computation books given in Tables 21, 22, 23 and 24, found the actual yield month by month for the past thirty-two years, and having modified the result to correspond with the future area of enlarged storage reservoirs and enlarged watershed, the next question is, what yield may safely be relied upon for the future? We can compute this best by assuming some uniform rate of draft, as, for example, 275 million gallons per day, and find how much water would have had to be borrowed from storage each month to help out the natural flow; and next, by summing up the consecutive drafting and refilling of storage, find the maximum storage volume required to maintain the assumed draft of 275 million gallons per day.

This has been done in the book from which Table 24 is a sample page, and in a similar manner we have computed the total storage that would have been required with uniform deliveries of 250 and 300 also for brief periods 225 and 325 million gallons daily. The result of these computations is given in the Depletion Diagram No. 3-B, page 29.

It is in this Depletion Diagram that we find the grand summing up of all the measurements of Croton Flow and Future Yield, heretofore described and presented in Tables 25–28, and it is by this Depletion Diagram that we obtain the most accurate measure of the resources of the Croton Watershed. It is mainly from a careful study of this diagram that I find the proof that 300 million gallons per day is the utmost steady draft that the Croton Watershed of 360.4 square miles above the New Croton Dam can sustain under rainfalls equivalent to those of the past thirty-two years; and from this in connection with certain experiences on other watersheds, and certain practical considerations about purity of water, that I form the opinion that 275 million gallons per day is as much as it can be prudently called upon to permanently supply.

The Depletion Diagram No. 3-B, page 29, shows just how much storage water would have been required in each month from 1868 to 1899 in order to supply various rates of flow, and it very clearly brings out the frequent cycles of low rainfall.

For example, we see that in a period of several successive years of low rainfall like 1870, 1871 and 1872, a steady draft of 275 million gallons per day, starting with storage reservoirs full on May 15, would have exhausted 46.2 billion gallons of storage by February 1, 1871. That the reservoirs

would have become less than half filled up again in the floods of March and April, and would have begun the dry season of 1871 on May 1 with a deficiency of 26 billion gallons in the reservoirs; that by October 1, 1871, the reservoirs would have been again depleted to 42 billion gallons below high-water mark, and that the fall rains and the spring floods would have failed to fill all the reservoirs again in March, and that they would still be down 8.5 billion gallons an May 1, when the dry season begins. They would be depleted to a total of about 23 billion gallons on November 1, and about February 1 they would all be full for the first time in two years and eight months.

We thus see that a reservoir capacity of 46 billion gallons would be needed to bridge over the low rainfall of 1870–72 enough to supply 275 million gallons per day, and thus that the present amount of reservoir volume would have been very nearly sufficient for 275 million.

We see similarly that in 1876–1877 a storage of 46.4 billion gallons would again have been required, and the reservoirs would have been below high-water mark for a year and nine months.

In 1880 and 1881 we see that a storage of 56 billion gallons would have been needed to bridge over the drought and supply 275 million gallons per day.

In 1891, 42 billion gallons storage would have been needed, and no water wasted for two years.

In 1899, had the draft been 275 million gallons per day, the reservoirs on December 31 would have shown a depletion of 36 billion gallons, while under the actual draft of about 226 million gallons per day, the reservoirs did show on December 31, 1899, a total depletion of 24 billion gallons.

In studying this diagram, we must keep in view the important practical consideration that the quality of water for drinking purposes may be injured by keeping a reservoir at a low level for too many years, thus permitting weeds and other organic matter to take root on its margins and shallow flowage.

If, for example, the three possible sites for new storage reservoirs in the Croton basin were improved, and the total reservoir capacity increased from the proposed 65 or 70 billions of 1902 to 85 billions, we see on the Depletion Diagram by following along the storage line computed for a draft of 325 million gallons per day through the years 1879 to 1888, that for the five successive years, 1881, 1882, 1883, 1884 and 1885, the waters from all the spring

floods, year after year, would have failed of refilling the reservoir to within 32 billion gallons of the spillway level. The reservoirs would have been 40 per cent. empty at their highest spring level in all these five years. We see further that not a gallon of water could waste over a spillway for nine years if this rate of draft of 325 million gallons per day were maintained and we can rest assured that no matter how great a storage reservoir capacity be built, the average yield cannot safely be forced to 325 million gallons per day.

Coming down next to a steady draft 300 million gallons per day, the 300 line in the year 1880 looks as if we might barely squeeze through, after practically using up almost the last gallon in the storage reservoirs; but in studying the rainfall record more closely, we note that one of the most severe rainfalls ever known was spread over the Croton Watershed in September, 1882. This filled the reservoirs and saturated the ground, so that under the severe drought of 1883, a heavy draft of water would have been felt here much less severely than on various other New England watersheds which did not receive this fortunate "cloud burst."

On the Sudbury, for example, which probably has been gauged more accurately than any other American watershed, the rainfall was less fortunate, and the drought of 1883 much more severely felt; and the same is true of certain other New England streams, of which we have accurate records. The long term run-off records of the Sudbury and Croton Watersheds average almost exactly the same.

For the twenty-five years from 1875 to 1899, inclusive, the average rainfall, run-off and percentage of rainfall collected, have been as follows:

Average Records for Twenty-five Years—1875 to 1899.

	INCHES OF RAINFALL.	INCHES OF RUN-OFF.	PERCENTAGE COLLECTED.
Sudbury Watershed, 75 square miles.....	45.83	22.48	49.15
Croton Watershed, 338 square miles.....	48.44	22.84	47.15
Croton greater than Sudbury.....	2.61	0.36	—2.00
Percent. excess. Croton over Sudbury.....	+5.70	+1.60	—4.07

Possibility of a Longer Series of Dry Years, or a Series of Drier Years, than Any Experienced in the Croton Watershed from 1868 to 1899.

So far as I am informed, there are no reliable records of rainfall in the vicinity of the Croton Watershed prior to 1868, from which we can judge if there has been a drier period than that of 1880 to 1883. A comparison of rainfall records in Boston, with records within the Sudbury Watershed, shows that records on the coast cannot always be relied upon to give the precipitation on a watershed thirty miles or more inland.

Taking Mr. de Varona's admirable arrangement of the records of rainfall at Brooklyn, N. Y., since 1826, given on page 48 of his report on additional water supply, in which he groups the years by threes and fives, and compares the average of these periods with the mean—

We find in the four years	1880–1883...	39.33 inches rainfall at Brooklyn.
" " " "	1892–1895...	37.47
" " " "	1861–1864...	37.11
" " " "	1846–1849...	38.95
" " " "	1837–1840...	38.65
While in the three years,	1881–1883...	38.86
" " " "	1893–1895...	37.38
" " " "	1862–1864...	34.26

Thus, so far as these Brooklyn records may serve, they show that the 1880–3 period was *not* the driest within the half century.

And the old records of rainfall at Boston and at Lowell indicate that in 1819–22 and 1834–7 there were droughts in New England more severe than that on the Sudbury in 1881–3. But this period of 1881–3 in the vicinity of Boston appears to have been the driest for 60 years.

Safe Limit of Croton Watershed, 275 Million Gallons per Day.

With the city normally using, say, 275 million gallons per day, it could, by choking off the gates on the distribution system so water would not rise above the first stories, and by vigorous efforts to stop waste, probably cut off 50 million gallons per day and worry along a month or two on 225 million gallons per day, very much as was done in the early part of 1891; but to empty the reservoirs entirely, and thus have to drop to the natural drought flow, plus the little water draining out of the reservoir margins, *would leave*

available probably scarcely 50 million gallons per day, which would at once shut down every factory and every steam and electric power plant in New York City, and cause an appalling amount of danger and suffering. The storage reservoirs existing in 1891 were entirely exhausted about November 15. The distribution reservoirs were but partly drawn. Fortunately, rains came promptly. Had these rains in that year been delayed one or two months, as is not infrequent, the disaster would have been frightful and beyond remedy. Although the thirty-two year record on the Depletion Diagram shows that 56 billion gallons of storage would have kept up the supply to 275 million gallons per day in the driest period, or in 1881–3, and notwithstanding that in 1902 there will be 65 billion gallons storage available without flashboards and without the distributing reservoirs, or a surplus of 9 billions above that needed for a 275-million gallon per day delivery, I would earnestly recommend that this extra 9 billion gallons—equivalent to thirty-two days' supply at 275 million gallons per day—be treated in the permanent organization of the system, as a reserve against great emergencies in addition to the 2 billion gallons in the distributing reservoirs.

Although the result of this recomputation of the yield of the Croton Watershed month by month for thirty-two years is about 10 per cent. smaller than the former official estimates, we must not forget that there is still 3 per cent. uncertainty in the present measurements, that we have taken average values for many factors which may actually be a little more or a little less. This 3 per cent. margin of uncertainty or lack of precision in our data amounts to 10 million gallons per day, and if, as is perhaps possible, this margin is 5 per cent., then 15 million gallons per day must be deducted from the 300 found for the minimum of the past thirty-two years, and some allowance must be made for an occasional error in management of storage and for the possibility of a more severe drought.

I therefore strongly recommend that 275 million gallons per day be regarded as the greatest allowable annual average draft from the new Croton Lake of 1902 under the best possible utilization of the storage, and with the possibility that once or twice in a century a succession of years of low rainfall may so curtail the supply that the average outflow for a 4-year period would be only 260 million gallons per day.

This remote or rare contingency when it comes can by foresight be limited to serious inconvenience instead of disaster by adopting some such rule as, for example, that when, on April 1, the reservoirs are found to have

refilled to only half their full capacity, then restrict the consumption until water is again plenty by throttling the pressure and by special vigilance in searching for waste, to, say, 225 million gallons for the ensuing seven months.

Turning from the question of the broad future to the immediate future, for example, the year 1901—

Total area Croton Watershed.....			338.8 sq. m.
" " of net land surface.....			327.6 " "
Per cent. of total watershed covered by water.....			3.31 %.
Proportion of storage available, 133 million gallons per square mile of net land.			
Total of storage available.....			43,390 mil. gals.

By the Sudbury diagrams, page 230, the corresponding safe flow is 659,000 gal. per sq. mile of land
659,000 × 327.6 = 216 million gals. per day.

If by flashboards storage is added to extent of.....	4,000?	mil. gals.	
If by lease of ponds not owned is added	855	" "	
If by emptying Central Park reservoirs there is added.....	1,000	" "	
The utmost total addition to storage in 1901 is.	5,855	" "	5,855 mil. gals.
The utmost total storage in 1901 is.....			49,245 " "

This total storage divided by the area gives 150 million gallons per square mile, and by the Sudbury diagrams the safe yield is 691,000 gallons per square mile;

691,000 × 327.6 =.....226 million gallons per day.

Referring now to the Depletion Diagram No. 3-B, page 29, we note that on January 31, 1881, 40 billion gallons of storage would have been exhausted by a daily draft of 225 million gallons per day, and that 48.8 billion gallons of storage would have been exhausted by a daily draft of 250 million

gallons per day. At the same time we note that in the severe drought of 1891 (see November, 1891, on Depletion Diagram No. 3-B), when the storage reservoirs within the watershed were all emptied, the actual amount of storage found available was about 1 billion gallons less than the nominal contents of the reservoirs. We see by proportion that 232 million gallons per day would be the utmost uniform daily draft obtainable from 43,390 gallons storage and the natural run of the stream in 1880, while with 49,245 gallons storage, the greatest uniform daily draft would have been 250 million gallons per day.

POSSIBILITY OF INCREASING CROTON YIELD BY ADDITIONAL STORAGE.

When the New Croton Dam is completed in 1902 to the height of 196 feet above sea level, without flashboards, the total available storage in the Croton Watershed controlled by the City of New York will amount to..... —per Table 18, pages 184–5.	66,590	million gallons.
If flashboards 4 feet high are added on new dam there will be added about.....	2,800	"
No plans have been proposed for overdrawing the New Croton Lake below the Aqueduct level.		
The special ponds, not owned by the City, drawn upon in 1870 and in 1880, were not drawn on in the severe drought of 1891, and we understand the City has no right to draw on them, but could perhaps make a trade for the water of a part of them at least in a great emergency. These aggregate	855	"
If all the water stored in Jerome Park Reservoir can be drawn out this would add.....	1,900	"
Thus outside of Central Park the greatest possible storage in reservoirs and natural ponds is.....	72,145	"

The storage practically available after 1902, tributary to the Croton supply, will therefore be somewhere between 65 and 70 billion gallons.

65 billions ÷ 344.3 square miles = 189 million gallons per square mile of net land.
70 billions ÷ 344.3 square miles = 203 million gallons per square mile of net land.
A yield of 275 million gallons per day is 800,000 gallons per square mile of net land.
A yield of 300 million gallons per day is 872,000 gallons per square mile of net land.

According to the best authorities it is not wise to carry the development of storage beyond the limits that will be reached by the development described above. Indeed, the above development is carried much beyond ordinary limits.

For example, Mr. F. P. Stearns in Rept. Mass. State Board of Health, 1890, p. 349: "The amount of water which can be made available from a watershed will not always depend on the quantity stored because considerations of quality require that levels of reservoirs should not be made to fluctuate too much and that water should not be drawn below high-water mark for too long a time."

Again, idem. p. 348: "The cases are very rare where more than 600,000 gallons per square mile (of net land) can be made available when it is necessary to store water in artificial reservoirs."

Mr. Desmond Fitzgerald, in Trans. Am. Soc. Civ. Engrs., 1892, vol. 2, p. 268, says: "No matter what the extent of the storage may be the effect of drafts beyond 600,000 gallons per square mile is to keep a storage reservoir below high water for several years. The importance of this warning cannot be too strongly impressed upon the engineer who is designing systems of storage." * * * "As it is undesirable to keep the water below high water for more than two years in succession it will be seen that no matter what the extent of the storage may be it is impracticable to secure more than about 750,000 gallons per square mile (of gross area) from a watershed containing 10 per cent. of water surface."

Mr. Stearns and Mr. Fitzgerald probably had the Sudbury in mind. On the Croton, with its 5.7 per cent. larger rainfall, and from a study of its actual run-off, as shown in the Depletion Diagram, I have ventured to extend the safe limit of draft to 275 million gallons per 24 hours, which is at the rate of about 763,000 gallons per square mile of gross area, and is about 800,000 gallons per day per square mile of net land.

The Depletion Diagram, Fig. 3, plainly shows that with this draft of 275 million gallons per day the water would have stood below high-water mark for two years at a time, and, on one occasion, 1870–1873, would have stood below the spillway level for 3 years, with no water wasted over Croton Dam meanwhile.

If more storage is to be constructed and used it will simply result in longer periods of depletion and will increase the chance of bad tastes in the water. It is my opinion that the development of storage, when the New Croton Dam is complete, will have been carried as far as it is profitable to carry it at present. Perhaps with filtration in the distant future it may be well to build one or two more reservoirs.

PROPOSED CROSS RIVER RESERVOIR.

Formal request was made some months ago by the Commissioner of Water Supply for an appropriation of about $980,000 for the purpose of acquiring lands and beginning the work of construction for a new storage reservoir in the Croton Watershed on the Cross river. A copy of this communication was shown to me in the Comptroller's office, with the suggestion that I consider its urgency.

It appears possible to reach a safe conclusion in the matter without an examination of the ground and without precise estimates of the cost of the necessary lands and structures.

I was informed orally by Mr. George W. Birdsall, Chief Engineer, Department of Water Supply, that this Cross River Reservoir would contain about 9 billion gallons and would probably cost complete for lands, construction and all attendant expenses, not far from $1,500,000, at a rough approxi-

mation, obviously the contents and the cost may be made to vary according to the height to which the dam is built.

This is apparently substantially the same site called "N" in the exploration for reservoir sites made in 1858, but is of very much larger volume than then proposed, according to the table in Wegman, page 79.

I find by computations presented below that this reservoir, if containing 9 billion gallons, would only add about 12 million gallons per day to the 275 million gallons already secured, and in so doing, would extend the period of depletion, in the low rainfall cycle, to a very undesirable extent.

It is stated in the communication from the Department of Water Supply that it would require about 5 years to secure the land and construct this reservoir, and while I think it probable that this time might be lessened under pressure, I do not think that water from that source could be obtained enough sooner than from the more abundant source that is surely needed, to be of any weight as an argument for the construction of this reservoir. This Cross river reservoir would not add to the present sources more than enough water to supply the natural increase of consumption coming from one year's growth. A much larger supply is imperative, and much more water for the money can be had as a part of this larger supply than by building a reservoir at Cross river. *I therefore recommend that its construction be postponed.* At some time in the distant future it may become advisable as a reserve against a longer drought or as a substitute for some reservoir that may temporarily develop a bad taste, or may become a very valuable adjunct after the future filtration plan, which, I believe, must come a generation hence, has been developed and the means for removing bad tastes arising from too long depletion thus avoided. Nine billion gallons would supply 275 million gallons per day for 32 days. The additional yield that would be gained by the construction of this Cross river reservoir is computed as follows:

First Method:

The length of period in which no water will run to waste while a draft of 275 million gallons per day is maintained in a series of years of low rainfall, like 1880 and 1881, and while a storage of 55,540 million gallons is being exhausted, is about 1 year and 11 months, or about 700 days (of which time the reservoirs would be undergoing depletion 400 days and refilling during about 300 days).

Had there been available additional storage of 9 billion gallons discharged at a uniform rate for this 700 days it would have augmented the supply $\frac{9,000,000,000}{700} = 13$ million gallons per day, or since the period required to refill would be longer with this additional reservoir space, the augmentation of flow gained by the 9 billions of additional storage would have to be spread over a slightly longer period and the average daily gain a little less than 13 million gallons.

We find by the depletion diagram other periods when with 275 million gallons per day drawn the aggregate reservoir capacity is even longer below high-water mark,

As from May, 1870, to February, 1873 = 2 years 8 months = about 970 days;

May, 1876, to about March, 1878 = nearly 20 months;

May, 1891, to April, 1893 = 1 year 11 months = about 700 days;

—so that the period of 700 days above taken, as that over which this added storage must be averaged, appears fair.

And it therefore appears fair to consider that this 9 billion gallons of storage, on top of what is now available, would only raise the safe yield of the watershed by 13 million gallons per day.

Second method for a check :

By the Sudbury Diagram (No. 49) of yield per square mile with various proportions of storage :

A watershed with.....	203	million gallons storage per square mile and 4.47% water may be relied upon for a safe yield of
	777,000	gallons per square mile of net land per day.
9 billions on 343. sq. m. =	26.2	million gallons storage per square mile added by Cross River.
Total storage.....	229.2	million gallons per square mile if Cross River is added.
The Sudbury diagram for this gives	810,000	gallons per square mile of net land.
Increase due Cross River.....	33,000	gallons per square mile of net land.

Or on whole area 33,000 gallons × 343 square miles = 11.3 million gallons per day added by the Cross River Reservoir.

From the above computation it appears that this proposed reservoir at Cross river, even though it be built with a dam high enough to hold 9,000 million gallons, would only increase the available supply 13 million gallons per day.*

Table No. 31, following, was prepared as a check on certain of the computations precedent to concluding that the greatest safe yield of the Croton Watershed in a cycle of years of low rainfall is 275 million gallons per day, also for determining the greatest reasonable development of storage.

It is of great interest to note the close agreement between the results found on the Sudbury Watershed, in part supplying Boston, and the new Croton figures. This gives us confidence in the application of these rates of watershed delivery to the Ten Mile and Housatonic Watersheds, and establishes confidence in the use of Diagram No. 48, page 230, as a safe and conservative basis for estimating the safe yield of the watersheds to be considered later for the future supply of Greater New York.

The 11th column of this Table No. 31 and also its 1st column afford confirmation that the figure of 275 million gallons per day for the safe yield is the largest that can be properly used.

Table No. 32, which follows Table No. 31, contains the same data previously given in Tables Nos. 27 and 29, rearranged in a form that is more

*It has been stated previously in this report that now with a total storage of 43,390 million gallons the safe yield would be 232 million gallons per day, while with 65,290 million gallons depletion the yield would be 300 million gallons per day. Whence it appears that increasing the storage 21,900 million gallons increases the safe yield 68 million gallons per day.

It may be asked why 9,000 millions increase of storage, as figured above, gives an increase of only 13 millions, whereas it would be in proportion to the case just cited, $\frac{9}{21}$ × 68 = 28 millions. The answer is, that because of the much longer period over which the storage has to be spread when higher rates of draft are maintained and larger storage volumes used, the proportionate gain is much lessened. The same fact is shown in diagram No. 48 by the far wider spacing between the lines of yield in the upper portion of diagram.

The 21,900 million gallons of storage first referred to has to be spread over a period of only about 320 days and thus averages $\frac{21900}{320}$ = 68 million gallons per day.

convenient and useful when considering the value of a stream for water-power, and was prepared as a step precedent to the water-power estimates of Appendix No. 8, and other similar estimates that will have to be made when the cost of diverting other possible sources of supply is considered. The average flow for each month per this Table No. 32, in comparison with that found for certain other streams, is presented in Figure 93.

STORAGE CAPACITY REQUIRED TO SUSTAIN A GIVEN RATE OF DRAFT, AS SHOWN BY THE CROTON RECORDS.

The principal data required by the hydraulic engineer in order to answer the question of how much is the safe daily yield of a given stream, like the Housatonic, Ten Mile, Esopus, Ramapo, or Wallkill, are:

1st. Area of the watershed.

2d. Annual rainfall in dry years and for a series of years.

3d. The possible storage capacity for flood waters to be let out in drought.

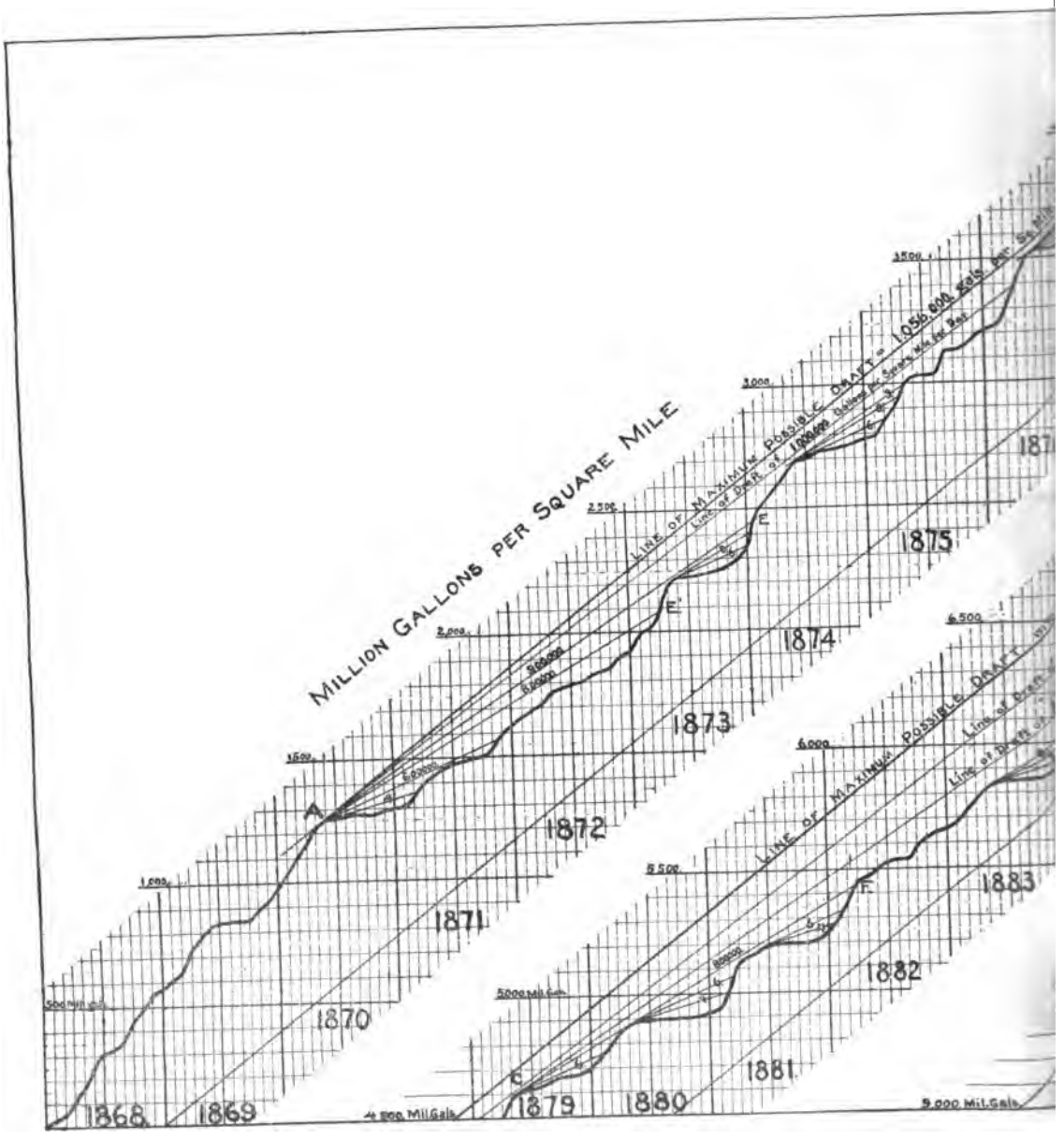
4th. The evaporation loss from these reservoirs between the period of storage and that of use.

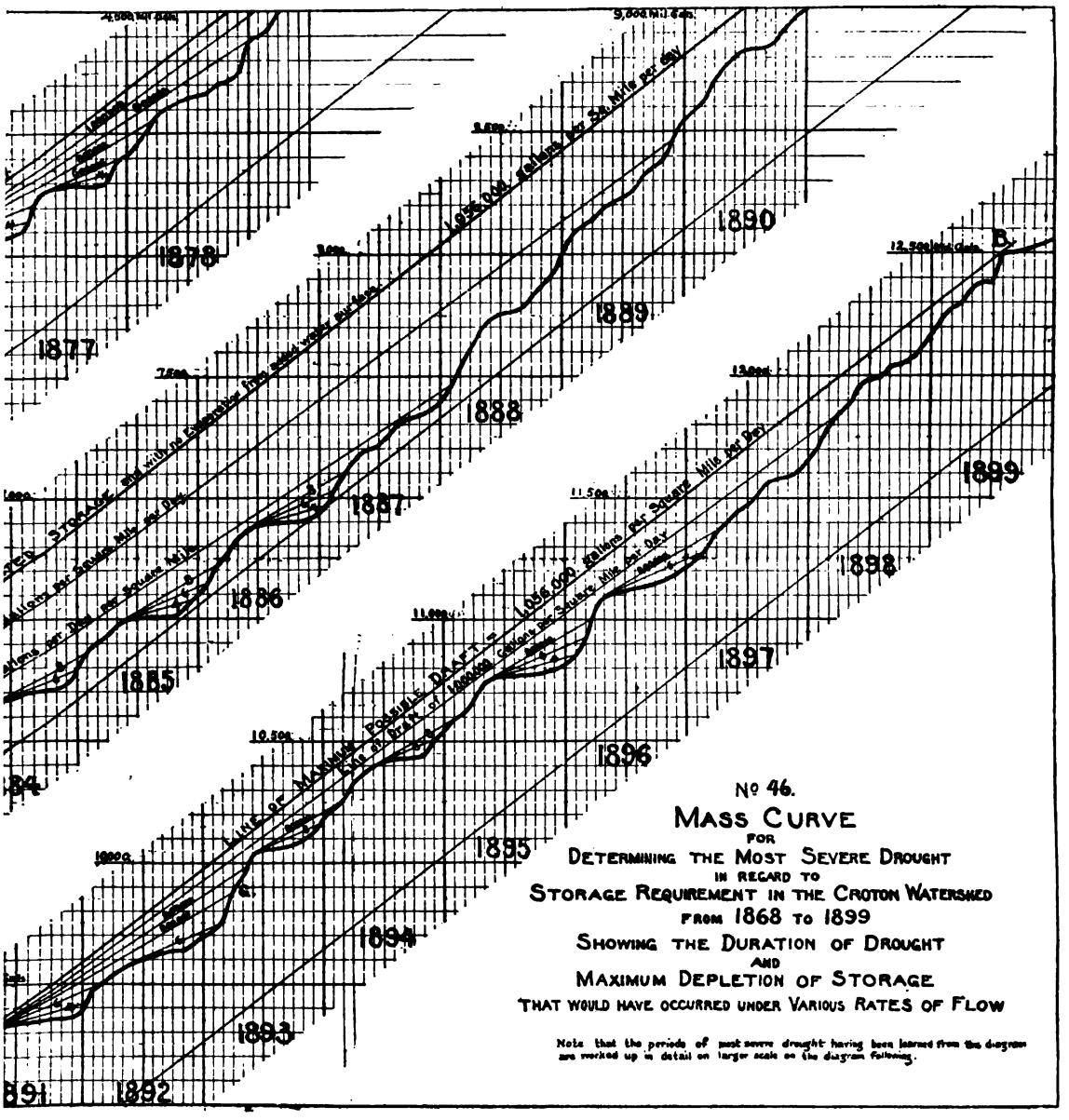
The final conclusive question to be answered from the above data is: How great a uniform flow, in gallons daily per square mile of watershed, can be sustained by the natural flow of this stream, augmented by a judicious use of the storage?

The available storage has a bearing on a correct answer hardly less important than that of watershed area. For example: without storage, the Croton Watershed of 360 square miles could not be relied on for more than 25 or 50 million gallons per day; while with 65 or 70 billion gallons of storage it can be relied on for 275 million gallons per day.

The records of Croton run-off for past thirty-two years are of utmost value for showing what may be derived from other streams similarly situated, receiving similar distribution of rainfall by a given development of storage, and no records heretofore obtained in this country are of greater value as a basis for estimating the safe supply that can be obtained from other possible sources for New York, and the only ones that approach them are the similar records derived by Fitzgerald and Stearns from the Sudbury River Records of the Boston Supply.

I have therefore prepared Diagram No. 48, which shows at a glance the storage volume required to safely maintain a given uniform rate of flow per square mile under various extents of water surface exposed to evaporation, similarly situated to the Croton.


MILLION GALLONS PER SQUARE MILE
LINE OF MAXIMUM POSSIBLE DRAFT = 1,050,000 GALS. PER SQ. MILE
Line of Draft of 1,000,000 Gals. per Sq. Mile
Line of Draft of 950,000 Gals. per Sq. Mile
Line of Draft of 900,000 Gals. per Sq. Mile
Line of Draft of 850,000 Gals. per Sq. Mile
LINE OF MAXIMUM POSSIBLE DRAFT
Line of Draft of
A
E
E'
F
F'
1868
1869
1870
1871
1872
1873
1874
1875
1879
1880
1881
1882
1883
4,000 Mil. Gals.
5,000 Mil. Gals.
9,000 Mil. Gals.



No 46.

MASS CURVE

FOR

DETERMINING THE MOST SEVERE DROUGHT

IN REGARD TO

STORAGE REQUIREMENT IN THE CROTON WATERSHED

FROM 1868 TO 1899

SHOWING THE DURATION OF DROUGHT

AND

MAXIMUM DEPLETION OF STORAGE

THAT WOULD HAVE OCCURRED UNDER VARIOUS RATES OF FLOW

Note that the periods of most severe drought having been learned from the diagram are worked up in detail on larger scale on the diagram following.

For confirmation and comparison I have added to Diagram No. 48 the results of a similar analysis made from the Sudbury River Records. These two curves confirm each other wonderfully for all ordinary rates of flow, but this very agreement prompts a word of caution as to assuming that there would be a similarly close agreement on a stream which, like the Esopus, is a flashy mountain stream requiring, doubtless, more storage to sustain a given unform rate of flow, notwithstanding that it may have a larger average annual rainfall; or to a stream like the Wallkill, which shows no hard, impervious bed rock at its dam sites, and which may possibly lose water by percolation through deep, pervious strata to a lower level.

The full analysis of these problems is very complex, but the results of an analysis are easily stated in a form that makes them very quickly available for use in the study of other similar problems, and Diagram No. 48, for example, sums up nearly all of the valuable information on river flow derived from thirty-two years of observation on the Croton run-off and twenty-five years of observation on the Sudbury.

The construction of such a diagram involves a large amount of computation, but once compiled, is well worth its cost. I am not aware that the method of construction has heretofore been described, and as the method that I have followed is in some respects new and is believed to be less laborious than that by which the similar Sudbury Diagram was prepared, it will be briefly outlined.

1st. Diagram No. 46 is prepared on a large open scale (about 3 times as large as the photo-engraving), by plotting the aggregate run-off of the stream from the beginning to the end of the record. The data are from Table No. 29 and are the measured run-offs, month by month, corrected for storage draft and the increased evaporation loss, and thus represent the mean monthly run-off of the stream at the New Croton Dam, with 4.47 per cent. of water surface exposed to evaporation in the watershed, divided by its gross area of 360.4 square miles; thus giving the natural flow per square mile. These mean daily flows for each month were multiplied by the number of days in the month and summed up on a Burkhardt reckoning machine, and the aggregate up to the end of each month plotted to form the main line or mass-curve of Diagram No. 46.

2d. Given this mass-curve, to find the greatest possible uniform rate of draft, regardless of practical limitation of storage capacity and regardless of the evaporation loss that would follow increase of storage. This is a little less than the arithmetic average of all the monthly averages, because these do not take account of leaving the storage in the same condition at the end as at the beginning of the term. The time average is found by drawing the line A B tangent to the two summits A and B, near the end of the mean curve, and is found to be at the rate of 1,056,000 gallons per square mile per day, which is plainly a much larger flow than could by any possibility be realized in practice.

3d. To find from this mass-curve the periods of greatest storage depletion and the dry year cycles of longest duration, we first lay off on the diagram straight lines at slopes corresponding on the scale of the diagram to various rates of uniform draft per square mile per day, and then

by straight-edge and triangle, rule parallel lines tangent to various summits, corresponding to full reservoirs, as A C D, and note where these again strike the mass-curve, as A E, C F, and D G. The vertical space between this straight line and the curve now represents the depletion of the reservoirs under the given rate of draft; and the distance from A to E represents the length of time that the reservoir would remain drawn down. For example: we see that by the scale of the diagram, with 4.47 per cent of water surface, to have maintained a rate of flow of 900,000 gallons per square mile per day, would have, by date of October 1, 1871, exhausted 190 million gallons of storage per square mile, and that the reservoirs would not have become refilled until January, 1874.

From a study of this Diagram No. 46, the various dry year cycles are readily picked out, and it is seen that the years from 1879 to 1884 furnish the most severe test of the capacity of the Croton watershed.

We therefore next prepare on a larger scale Diagram No. 47, covering only this period from 1879 to 1884. Diagram No. 46 was for the condition of 4.47 per cent. of water surface, but to apply the Croton determinations to other watersheds, it is important to consider various percentages of evaporating surface in ponds and reservoirs. The most convenient means for doing this is by the aid of Table No. 20, in which the effect upon the yield, due to the increased evaporation, from covering one square mile of land surface by water, as when filling a new storage reservoir, has been carefully considered. During the actual measurements of Croton run off between the years 1879 and the year 1884, the water area of all ponds and reservoirs in the Croton watershed was actually 2.03 per cent., or say 2 per cent., and therefore the mass-curve for the aggregate run-off, beginning May 1, 1879, and multiplying the monthly means of Table No. 26 by the number of days in each month and summing them up, is plotted directly.

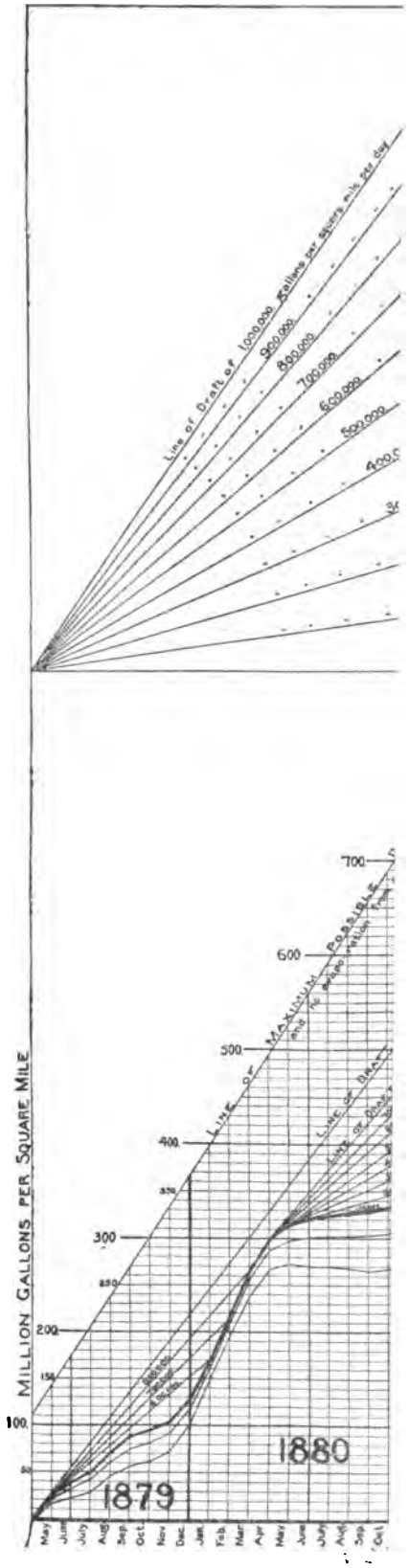
For other percentages of water surface in the watershed, the reduction in yield, due to the increased evaporation as per table No. 20, was computed and summed up and the aggregate loss up to the end of each month laid off in the diagram by measuring down from the 2 per cent. curve (or actually this computation was made on the reckoning machine and plotted directly). To avoid confusion on the reduced scale of the reproduction of Fig. 47, several of the intermediate curves are omitted and the series of sloping tangents is reproduced only for the 2 per cent. curve.

By now noting the point of maximum depletion and the extent of this depletion as measured on the scale of the diagram, as at K L, we note that under a uniform draft of 500,000 gallons per day per square mile, the maximum depletion of storage would have occurred in February, 1881, and would have amounted to 77 mil. gals. per sq. m., with 2 per cent. of water surface,
or " 87 " 7.5 "
or " 101 " 15 "
and we further see that the reservoir would have continued below high-water mark for 9 months on a watershed with 2 per cent. of water surface, or from June 1, 1880, to March 1, 1881.

By similar measurements on Diagram No. 47 we obtain the quantities for plotting upon Diagram No. 48.

Attention has elsewhere been called to the abnormal rainfall of September, 1882, upon the Croton Watershed, and a study of this in comparison with rainfall elsewhere at that time indicates that this was something so unlikely to be repeated at another such critical time as to be an unsafe reliance for estimates of the safe future yield of this or other watersheds. The shape of the mass-curve plainly shows the abnormal condition prevailing at that time.

Upon the Sudbury no such abnormal condition happened to prevail, and the Sudbury records appear to be the safer and more reliable data for


Line of Draft of 1,000,000 Gallons per Square mile per day
900,000
800,000
700,000
600,000
500,000
MILLION GALLONS PER SQUARE MILE
700
600
500
400
300
200
100
1879
1880
May
June
July
Aug
Sep
Oct
Nov
Dec
Jan
Feb
Mar
Apr
May
June
July
Aug
Sep
Oct

high rates of yield; but to see how far we could allow for the effect of this abnormal rainfall of September, 1882, I have ventured some readjustment and shown the result on Figure 47.

The readjustment is as follows:

(a). It appears certain that an extraordinary rainfall did occur upon the Croton in September, 1882, because the several rain-gauges, miles apart, and measured by different observers, all agree reasonably in showing a total precipitation for the month of September of..... 14.63 inches.
and comparison with 30 other Septembers shows the excess to have been about..... 12.00 "

(b). The record of depths over Croton Dam show
that in 8 days following this most severe storm..... 14,867 mil. gals. of water wasted over the dam,
and that in the next 11 days..... 3,350 " " "
A total of..... 18,217 " " "

This is equivalent to 54,200,000 gallons of waste per square mile of watershed.

One inch of water over one square mile amounts to ..... 17.4 million gallons.

Wherefore it appears that $\frac{54.2}{17.4}$..... 3.05 inches
of this rain ran off directly within 19 days.

The reservoir records show that..... 0.39 "
of this rain went to re-fill reservoirs.

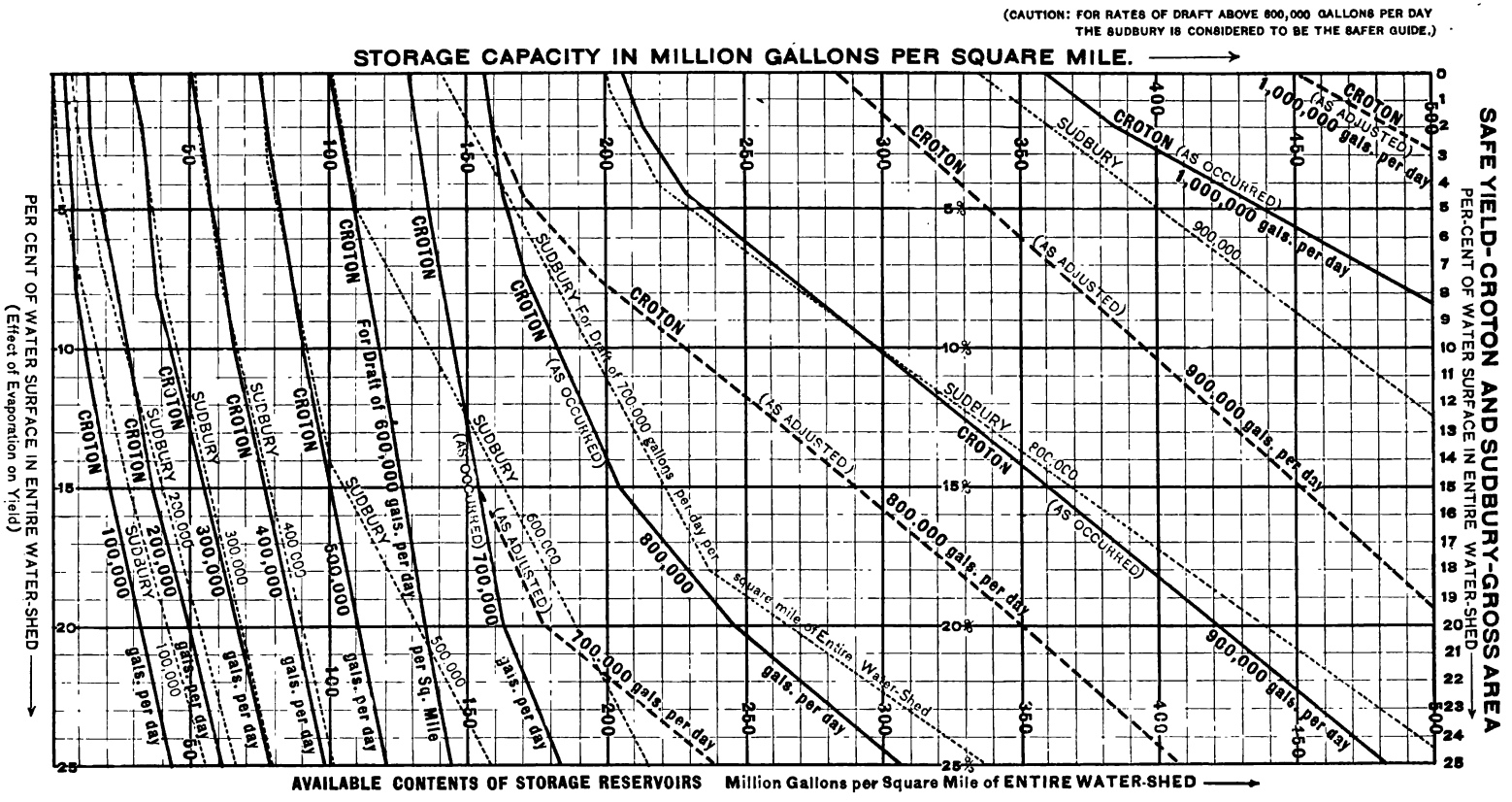
Total rainfall running off within 19 days ..... 3.44 "

In the long run, 50% of the rainfall re-appears as stream flow, but much of the water which sinks into the ground and slowly percolates toward the bottom of the valley may be months or years in re-appearing in the stream.

The total rainfall of 1882, mainly because of this abnormal September yield, was
52.35—47.83 ..... 4.52 inches.
more than the average rainfall for the 32 years.

After considering the probabilities from various points of view, it appears that we are at least justified in dropping the mass-curve bodily, to the extent of 100 million gallons per square mile, which corresponds to $\frac{100}{17.4}$=5.75 inches rainfall,

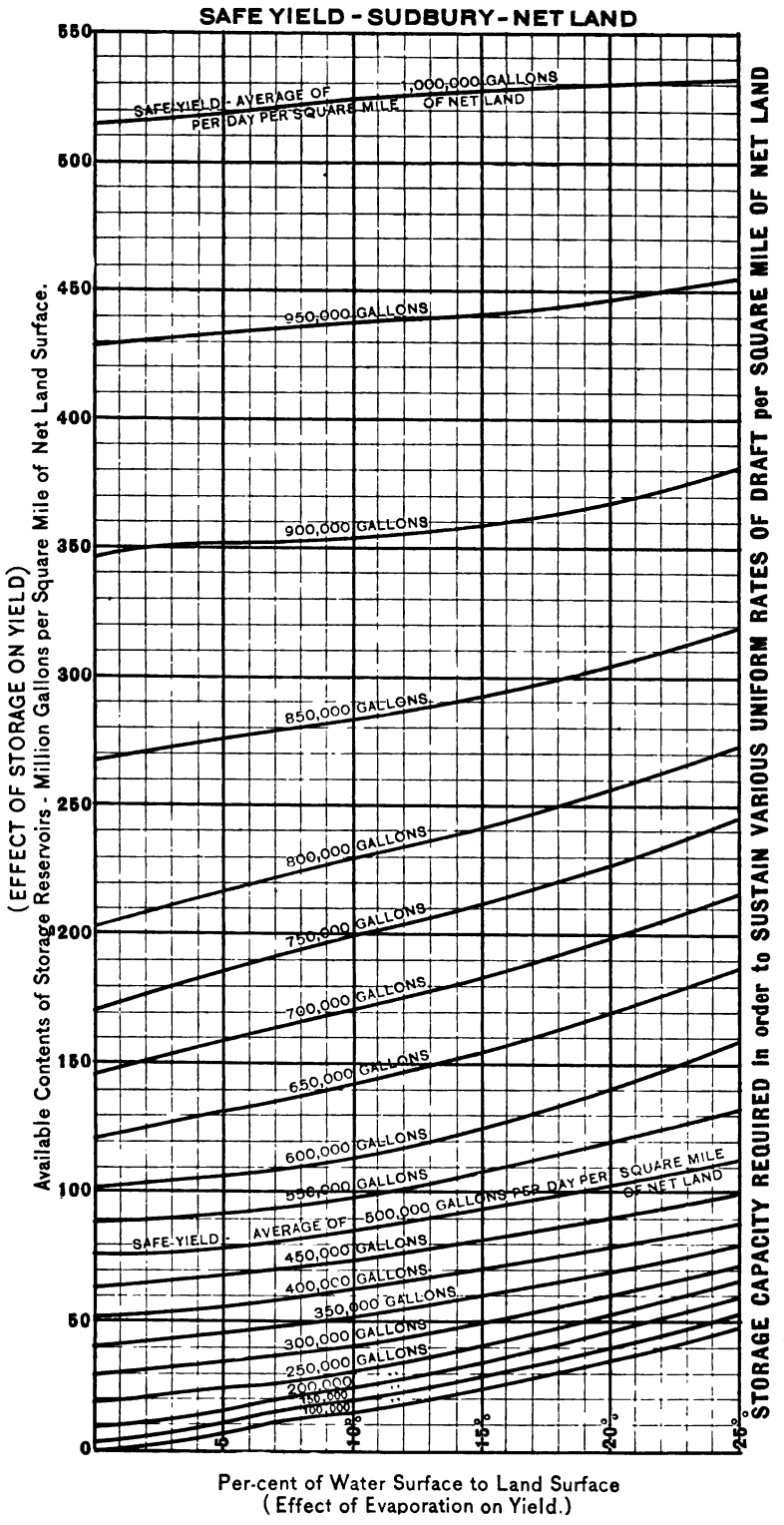
and therefore between September 1st and November 1st and for all the rest of the period shown on Diagram No. 47, I have dropped the curves 100 million gallons in height; and while the exact measure of this correction is unsatisfactory and uncertain, it certainly gives a safer and more probable basis for estimates than to let it stand exactly as located by the observations. It will be noted that this question does not affect the result at all for any rate of flow less than 700,000 gallons per square mile, 15% of water surface, and that no doubt is therefore cast over the reliability of Diagram No. 48, up to the limit of any ordinary storage development for purposes of water supply. For computing yields and storage requirements for larger rates of flow than this, I should always make use of the more conservative Sudbury records.


(CAUTION: FOR RATES OF DRAFT ABOVE 600,000 GALLONS PER DAY THE SUDBURY IS CONSIDERED TO BE THE SAFER GUIDE.)
STORAGE CAPACITY IN MILLION GALLONS PER SQUARE MILE.
SAFE YIELD-CROTON AND SUDBURY-GROSS AREA
PER-CENT OF WATER SURFACE IN ENTIRE WATER-SHED
PER CENT OF WATER SURFACE IN ENTIRE WATER-SHED
(Effect of Evaporation on Yield)
AVAILABLE CONTENTS OF STORAGE RESERVOIRS Million Gallons per Square Mile of ENTIRE WATER-SHED
CROTON
SUDBURY
(AS OCCURRED)
(AS ADJUSTED)
1,000,000 gals. per day
900,000 gals. per day
800,000 gals. per day
700,000 gals. per day
600,000 gals. per day
500,000 gals. per day
400,000 gals. per day
300,000 gals. per day
200,000 gals. per day
100,000 gals. per day
For Draft of 600,000 gals. per day per Square Mile of Entire Water-Shed
For Draft of 700,000 gallons per day per square mile of Entire Water-Shed

STORAGE CAPACITY REQUIRED in order to SUSTAIN VARIOUS RATES OF DRAFT from ONE SQUARE MILE OF WATER-SHED CONTAINING VARIOUS PERCENTAGES OF WATER SURFACE

Based on greatest droughts that have occurred on the Croton River from 1868 to 1899 and on the Sudbury River from 1875 to 1890.

Fig. 48



for Water sheds having same Monthly distribution of Rainfall and evaporation as Sudbury River in Massachusetts.

(Based on 25 years measured run-off from the Sudbury, including the Cycle of dry years 1879-1883 which appear to have had the smallest rainfall of any equal period during the past half Century, although perhaps not so severe a drought as in the year 1822.)

From data prepared by F. P. Stearns, C. E. This differs from Fig. 48 in basing the yield on the NET land surface while Fig. 48 bases the yield on the total Water Shed area, including water surface and land.

No. 49

TABLE No. 31.

Comparison of Yield of Croton and Sudbury Watersheds. Duration and Yield of Driest Periods.

Average Daily Natural Flow, Exclusive of Water Drawn from Storage, from Sudbury and Croton Watersheds; Compared for Different Periods Varying from One Month to Sixteen Years, Selecting in Each Case the Driest Continuous Period of a Given Duration.

Yield in Million Gallons Daily from Sudbury Experience applied to Croton Area 360.4 Square Miles.	Sudbury Watershed. 75.2 Sq. Miles after 1880. 2.31 Per Cent. of Water Surface, 1879-86. Dates inclusive. From records of 21 years (containing the most severe drought shown by rainfall records for 60 years). From	To	Actual Yield in Gallons per Day per Square Mile of Gross Area.	Length of Period.	Croton Watershed. Gallons per day per Square Mile of Gross Area. Actual Past Conditions Natural Flow 338.8 Square Miles.	Future conditions of Evaporation 360.4 Square Miles, 4.47% water.	Dates inclusive. From record of 32 years. From	To	Yield in Million Gallons Daily. Actual Past Conditions 338.8 Square Miles Gross Area.	Future Conditions of Evaporation 360.4 Square Miles, 4.47 % Water. Gross Area.
1	2	3	4	5	6	7	8	9	10	11
16	Sept., 1884	.....	*44,000	1 Month.	†	.....	Sept., 1881	.....	...	....
23	" "	Oct., 1884	64,000	2 Months.	....	.....	Aug., "	Sept., 1881	...	....
34	July, 1883	Sept, 1883	95,000	3 "	.....	... .	July, "	" "	...	....
43	" "	Oct., "	118,000	4 "	.....	.....	" 1880	Oct., 1880	...	....
47	June, 1880	" 1880	131.000	5 "	118,000	85,000	June, "	" "	40	31
52	" "	Nov., "	143,000	6 "	141,000	120,000	" "	Nov., "	49	43
53	" "	Dec., "	147,000	7 "	155,000	138,000	" "	Dec., "	52	50
65	" "	Jan., 1881	181,000	8 "	190,000	172,000	" "	Jan., 1881	65	62
79	May, "	" "	219,000	9 "	222,000	204,000	May, "	" "	75	74
112	April, "	" "	312,000	10 "	309,000	290,000	April, "	" "	105	104
147	Mar., "	" "	409,000	11 "	423,000	404,000	Mar., "	" "	143	145
179	" "	Feb., "	497,000	1 Year.	531,000	506,000	Feb., "	" "	176	183
248	Feb., 1882	Jan., 1884	687,000	2 Years.	744,000	733,000	Dec., 1879	Nov., "	252	264
275	Mar., 1880	Feb., 1883	764,000	3 "	817,000	797,000	Jan., "	Dec., "	277	287
265	Feb., "	Jan., 1884	735,000	4 "	839,000	822,000	Oct., "	Sept., 1883	282	296

277	Jan., 1879	Dec., 1883	**769,000**	**5**	"	**848,000**	829,000	Jan., "	Dec., "	287	**299**
290	Oct., "	Sept., 1885	**803,000**	**6**	"	**861,000**	853,0 0	Oct., "	Sept., 1885	292	**307**
302	Jan., "	Dec., "	**839,000**	**7**	"	**895,000**	885,000	Nov., "	Oct., 1886	303	**319**
314	" "	" 1886	**870,000**	**8**	"	**898,000**	879,000	Jan., "	Dec., "	304	**317**
325	" "	" 1887	**902,000**	**9**	"	**937,000**	913,000	Oct., 1876	Sept., 1835	314	**329**
340	April, 1878	Mar., 1888	**944,000**	**10**	"	**940,000**	925,000	May, "	April, 1886	318	**333**
349	Jan., 1875	Dec., 1885	**968,000**	**11**	"	**940,000**	925,000	June, "	May, 1887	318	**333**
353	" "	" 1886	**978,000**	**12**	"	**977.000**	965,000	Dec., 1875	Nov., "	331	**348**
357	" "	" 1887	**991,000**	**13**	"	**973,000**	959.000	June, 1874	May, "	329	**346**
374	May, 1879	April, 1893	**1,038,000**	**14**	"	**997,000**	982,000	" 1873	" "	337	**354**
370	June, "	May, 1894	**1,028,000**	**15**	"	**979,000**	961,000	Oct., 1870	Sept., 1885	332	**346**
366	July, "	June, 1895	**1,015,000**	**16**	"	**978,000**	967,000	Nov., "	Oct., 1886	331	**348**

* On the Sudbury, with the completion of larger storage basins, giving increased evaporating surface, there have been brief periods when the watershed as a whole has given a negative yield, but of course, at such times, there has been some water running in the river, less than enough, however, to balance the evaporation loss on reservoirs.

† Minimum Croton yield for less than 4 months not included because of the anomalous yield of September, 1881, when, taking storage records as they stand, the computation shows the amount drawn from storage (plus the loss from reservoirs by evaporation) was greater than the quantity carried off by aqueduct, and by omitting to compute and apply the loss by evaporation to the discharge from storage, the record presents a negative flow. This negative does not necessarily mean that no water would have flowed out of the lower end of the watershed if no storage had been drawn, but means that the watershed as a whole would have lost, or, in other words, that the lowering of level of pond surfaces by evaporation alone would have more than offset any delivery from Croton river into Croton lake.

TABLE No. 32.

Natural Flow of Croton River at Site of New Croton Dam (with Evaporating Surface as in 1902).

With months arranged in order of dryness.

Measurements of the given year compensated to apply to conditions of 1902 by adding a pro rata allowance for increase of watershed and deducting an allowance for the increased evaporating surface of New Reservoir.

Drainage area = 360.4 square miles, of which 16.1 square miles, or 4.47 per cent., is water surface.

	1868.			1869.			1870.			1871.			1872.		
	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.
Driest	Feb.	4,520	.67	Sept.	570	.08	Sept.	420	.06	Sept.	2,850	.41	July	2,850	.40
2d driest	July	5,700	.79	Aug.	1,550	.22	Oct.	1,490	.21	July	3,470	.48	Oct.	5,890	.82
3d "	Dec.	9,080	1.26	July	2,980	.41	Aug.	2,230	.31	Jan.	3,500	.49	June	5,880	.84
4th "	Aug.	11,190	1.55	June	6,630	.94	July	2,260	.31	Aug.	4,370	.61	May	6,230	.86
5th "	Jan.	11,220	1.55	Feb.	10,950	1.68	Nov.	3,690	.53	June	7,110	1.02	Sept.	6,120	.88
6th "	Oct.	16,460	2.28	Nov.	11,820	1.69	Dec.	3,720	.52	May	10,660	1.48	Dec.	8,150	1.13
7th "	June	17,760	2.54	Jan.	12,770	1.77	June	3,810	.55	Apr.	10,530	1.51	Aug.	8,770	1.21
8th "	Apr.	20,730	2.97	Oct.	14,820	2.05	May	9,300	1.29	Oct.	11,040	1.53	Mar.	9,270	1.28
9th "	Nov.	22,790	3.19	May	15,280	2.12	Mar.	19,410	2.69	Dec.	17,960	1.79	Feb.	9,510	1.41
10th "	Mar.	24,090	3.34	Dec.	18,450	2.55	Jan.	22,410	3.10	Feb.	12,290	1.88	Jan.	11,690	1.62
11th "	Sept.	30,120	4.31	Apr.	18,450	2.64	Apr.	24,630	3.52	Mar.	19,250	2.67	Nov.	15,000	2.15
Wettest	May	32,740	4.53	Mar.	30,780	4.26	Feb.	24,050	3.69	Nov.	21,060	3.01	Apr.	16,800	2.40
Total for year	.....	205,900	....	.....	145,050	....	.....	117,450	....	.....	119,090	....	.....	105,160	....
Average per square mile	.....	.....	2.42	.....	.....	1.71	.....	.....	1.38	.....	.....	1.40	.....	.....	1.24
Total for 6 dry months	184	58,170	....	180	34,500	....	184	13,810	....	184	31,960	....	183	35,120	....
Average per square mile, 6 dry months	.....	.....	1.36	.....	.....	.82	.....	.....	.32	.....	.....	.75	.....	.....	.82

Natural Flow of Croton River at Site of New Croton Dam (with Evaporating Surface as in 1902)—Continued.

	1873.			1874.			1875.			1876.			1877.		
	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.
Driest	June	2,040	.29	Sept.	1,500	.21	July	1,580	.22	July	1,150	.16	July	930	.13
2d driest	Sept.	2,280	.33	Nov.	3,690	.53	June	2,250	.32	Aug.	1,240	.17	Sept.	990	.14
3d "	July	2,360	.33	Aug.	4,340	.60	Jan.	3,260	.45	Sept.	1,590	.23	Aug.	1,150	.16
4th "	Aug.	3,780	.52	June	4,290	.61	Sept.	4,170	.60	Oct.	1,910	.27	June	2,640	.38
5th "	Oct.	7,780	1.08	Oct.	4,370	.61	Oct.	4,400	.61	Dec.	2,700	.37	May	4,180	.58
6th "	Nov.	9,540	1.36	Dec.	5,800	.80	Dec.	10,170	1.41	June	2,880	.41	Jan.	4,960	.69
7th "	Feb.	9,160	1.40	July	7,440	1.03	May	12,460	1.73	Nov.	4,440	.64	Oct.	6,700	.93
8th "	May	11,220	1.55	May	17,020	2.36	Nov.	12,330	1.76	Jan.	8,180	1.13	Feb.	8,480	1.30
9th "	Dec.	19,130	2.65	Feb.	15,740	2.41	Mar.	17,790	2.46	May	10,380	1.44	Dec.	11,160	1.55
10th "	Mar.	22,290	3.09	Mar.	18,260	2.53	Feb.	24,000	3.68	Feb.	20,650	3.06	Apr.	15,930	2.28
11th "	Jan.	24,300	3.36	Apr.	19,770	2.83	Aug.	31,530	4.37	Apr.	34,710	4.97	Nov.	26,400	3.78
Wettest	Apr.	39,990	5.72	Jan.	52,300	7.24	Apr.	31,590	4.52	Mar.	39,490	5.47	Mar.	40,430	5.60
Total for year	.....	153,870	....	.....	154,520	....	.....	155,530	....	.....	129,330	....	.....	123,920	....
Average per square mile	.....	.....	1.81	.....	.....	1.82	.....	.....	1.83	.....	.....	1.52	.....	.....	1.46
Total for 6 dry months	183	27,780	....	183	23 990	....	184	25.830	....	184	11,480	....	184	14,850	....
Average per square mile, 6 dry months	.....	.....	.65	.....	.....	.56	.....	.....	.60	.....	.....	.27	.....	.....	.35

With Months Arranged in Order of Dryness.

Natural Flow of Croton River at Site of New Croton Dam with Evaporating Surfaces as in 1902—Continued.

	1878.			1879.			1880.			1881.			1882.		
	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.
Driest	Aug.	3,160	.44	Oct.	2,170	.30	Aug.	590	.08	Sept.	—810	—.12	Aug.	310	.04
2d driest	July	3,690	.51	Nov.	3,420	.49	Sept.	750	.11	Aug.	190	.03	July	2,570	.37
3d "	Oct.	5,300	.73	July	3,840	.53	Oct.	900	.13	July	2,010	.28	Nov.	5,430	.78
4th "	June	7,920	1.13	June	4,380	.63	July	1,240	.17	Oct.	2,510	.35	Apr.	7,380	1.05
5th "	May	8,460	1.17	Sept.	5,820	.83	June	1,200	.17	Nov.	3,330	.48	Dec.	7,560	1.05
6th "	Apr.	9,000	1.29	Aug.	7,100	.98	Dec.	2,730	.38	Jan.	5,180	.72	June	8,730	1.25
7th "	Nov.	12,510	1.79	Jan.	8,000	1.11	Nov.	3,180	.45	May	7,040	.97	May	12,340	1.71
8th "	Sept.	13,170	1.88	Dec.	9,450	1.31	May	4,620	.64	June	8,820	1.26	Oct.	12,560	1.74
9th "	Jan.	16,400	2.27	May	9,490	1.31	Apr.	11,400	1.63	Apr.	10,200	1.46	Jan.	16,450	2.28
10th "	Mar.	21,330	2.95	Feb.	15,290	2.34	Jan.	15,560	2.15	Dec.	12,860	1.78	Sept.	20,460	2.93
11th "	Feb.	21,500	3.30	Mar.	23,560	3.26	Mar.	16,990	2.35	Feb	28,700	4.40	Mar.	28,860	4.00
Wettest	Dec.	45,010	6.25	Apr.	28,080	4.02	Feb.	17,260	2.56	Mar.	38,010	5.28	Feb.	26,910	4.13
Total for year	.....	167,450	....	.....	120,600	....	.....	76,420	....	.....	118,040	....	.....	149.540	....
Average per square mile	.....	.....	1.97	.....	.....	1.42	.....	.....	.90	.....	.....	1.39	.....	.....	1.76
Total for 6 dry months	184	37,530	....	183	26,730	....	184	7,410	....	184	12,410	....	183	31,980	....
Average per square mile, 6 dry months	.....	.....	.88	.....	.....	.63	.....	.....	.17	.....	.....	.29	.....	.....	.75

With Months Arranged in Order of Dryness.

Natural Flow of Croton River at Site of New Croton Dam with Evaporating Surfaces as in 1902—Continued.

	1883.			1884.			1885.			1886.			1887.		
	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.
Driest	Sept.	—150	—.02	Oct.	2,010	.28	Sept.	300	.04	Sept.	330	.05	Sept.	4,500	.64
2d driest	Aug.	620	.09	June	2,700	.39	July	840	.12	Oct.	960	.13	Nov.	5,580	.80
3d "	July	1,800	.25	Sept.	2,790	.40	June	1,560	.22	Aug.	1,980	.27	Oct.	5,770	.80
4th "	Oct.	2,980	.41	July	4,500	.62	Aug.	2,170	.30	July	2,480	.34	June	6,060	.87
5th "	June	3,060	.44	Aug.	6,260	.87	Oct.	2,640	.37	June	3,810	.55	May	6,630	.92
6th "	Dec.	4,150	.58	Nov.	6,510	.93	May	8,180	1.13	Nov.	6,060	.87	Dec.	14,200	1.97
7th "	Nov.	4,800	.69	May	9,980	1.38	Mar.	11,070	1.53	Dec.	8,710	1.21	July	14,790	2.05
8th "	Jan.	6,510	.90	Jan.	13,550	1.88	Dec.	13,790	1.91	May	11,320	1.57	Jan.	18,200	2.52
9th "	May	6,850	.95	Apr.	16,560	2.37	Apr.	14,220	2.04	Mar.	13,730	1.90	Apr.	18,600	2.66
10th "	Apr.	14,880	2.13	Dec.	23,090	3.20	Nov.	14,910	2.13	Jan.	20,210	2.80	Aug.	19,030	2.76
11th "	Mar.	16,120	2.23	Mar.	28,490	3.95	Feb.	14,110	2.16	Apr.	26,220	3.75	Mar.	20,890	2.89
Wettest	Feb.	19,350	2.97	Feb.	31,840	4.71	Jan.	24,550	3.40	Feb.	27,520	4.22	Feb.	28,980	4.44
Total for year	.....	80,970	....	.....	148,280	....	.....	108,340	....	.....	123,330	....	.....	164,130	....
Average per square mile	.....	.....	.95	.....	.....	1.74	.....	.....	1.27	.....	.....	1.45	.....	.....	1.93
Total for 6 dry months	184	12,460	....	183	2,470	....	184	15,699	....	183	15,620	....	183	42,740	....
Average per square mile, 6 dry months	.....	.....	.29	.....	.....	.58	.....	.....	.37	.....	.....	.37	.....	.....	1.00

With Months Arranged in Order of Dryness.

Natural Flow of Croton River at Site of New Croton Dam (with Evaporating Surfaces as in 1902)—Continued.

	1888.			1889.			1890.			1891.			1892.		
	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.
Driest	July	1,830	.25	June	7,380	1.06	Aug.	2,600	.36	July	1,240	.17	Oct.	1,300	.18
2d driest	Aug.	5,890	.82	July	8,590	1.19	July	3,810	.53	Sept.	1,260	.18	Sept.	2,490	.37
3d "	June	7,140	1.02	May	9,080	1.26	June	7,980	1.14	Oct.	1,360	.19	July	3,160	.44
4th "	Oct.	14,260	1.97	Oct.	10,570	1.46	Dec.	10,040	1.39	Aug.	1,700	.24	Aug.	5,800	.80
5th "	May	14,820	2.05	Mar.	11,380	1.58	Nov.	11,970	1.71	June	3,030	.43	June	6,180	.88
6th "	Nov.	17,910	2.56	Sept.	12,420	1.78	Sept.	12,090	1.73	Nov.	3,990	.57	Apr.	7,860	1.12
7th "	Sept.	19,020	2.72	Feb.	13,160	2.02	Jan.	13,110	1.82	May	4,560	.63	May	8,680	1.20
8th "	Jan	23,530	3.26	Apr.	14,280	2.04	May	14,600	2.02	Dec.	10,110	1.40	Dec.	9,300	1.29
9th "	Mar.	27,130	3.76	Aug.	23,400	3.24	Apr.	18,390	2.63	Apr.	15,720	2.25	Feb.	9,830	1.46
10th "	Feb.	27,430	4.06	Jan.	25,790	3.57	Oct.	19,560	2.71	Mar.	27,200	3.77	Nov.	10,350	1.48
11th "	Apr.	28,410	4.07	Dec.	26,910	3.73	Feb.	18,310	2.81	Feb.	33,710	5.17	Mar.	13,330	1.85
Wettest	Dec.	31,090	4.31	Nov.	31,140	4.46	Mar.	27,530	3.81	Jan.	40,830	5.65	Jan.	30,320	4.20
Total for year	.....	218,460	....	.....	194,100	....	.....	159,990	....	.....	144,710	....	.....	108,600	....
Average per square mile	.....	.....	2.56	.....	.....	2.28	.....	.....	1.88	.....	.....	1.70	.....	.....	1.27
Total for 6 dry months	184	61,940	....	184	59,420	....	183	48,490	....	183	12,580	....	183	26,790	....
Average per square mile, 6 dry months	.....	.....	1.45	.....	.....	1.39	.....	.....	1.14	.....	.....	.30	.....	.....	.63

With Months Arranged in Order of Dryness.

Flow of Croton River at New Croton Dam—Continued.

	1893.			1894.			1895.			1896.		
	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.
Driest	July	1,670	.23	July	1,270	.18	Sept.	1,050	.15	Aug.	3,070	.43
2d driest	Sept.	2,580	.37	Aug.	1,330	.19	Aug.	1,360	.20	May	3,100	.43
3d "	Aug.	3,720	.52	Oct.	3,220	.45	June	1,650	.24	Sept.	3,510	.50
4th "	June	5,040	.72	Sept.	3,390	.49	Oct.	1,950	.27	July	3,910	.54
5th "	Oct.	7,440	1.03	June	7,740	1.11	July	2,670	.37	June	4,980	.71
6th "	Jan.	9,360	1.30	Jan.	9,830	1.36	Nov.	4,710	.67	Oct.	5,110	.71
7th "	Nov.	11,610	1.66	May	10,110	1.40	Feb.	5,910	.91	Dec.	8,520	1.18
8th "	Apr.	19,470	2.79	Feb.	11,960	1.83	May	6,980	.97	Nov.	10,770	1.54
9th "	Feb.	20,380	3.12	Apr.	13,200	1.89	Dec.	7,870	1.09	Jan.	12,770	1.77
10th "	Dec.	23,650	3.28	Dec.	15,190	2.10	Jan.	19,470	2.70	Apr.	17,490	2.50
11th "	May	34,070	4.87	Nov.	20,610	2.95	Mar.	21,020	2.91	Feb.	26,160	3.87
Wettest	Mar.	40,800	5.65	Mar.	28,830	3.99	Apr.	23,430	3.35	Mar.	44,890	6.22
Total for year	.....	179,790	....	.....	126,680	....	.....	98,070	....	.....	144,280	....
Average per square mile	.....	.....	2.11	.....	.....	1.49	.....	.....	1.15	.....	.....	1.69
Total for 6 dry months	184	29,810	....	184	26,780	....	183	13,390	....	184	23,680	....
Average per square mile, 6 dry months	.....	.....	.70	.....	.....	.63	.....	.....	.31	.....	.....	.56

With Months Arranged in Order of Dryness.

Natural Flow of Croton River at Site of New Croton Dam (with Evaporating Surfaces as in 1902)—Continued.

	1897.			1898.			1899.			Average of 32 Years.
	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	Month.	Total Million Gallons.	Cubic Feet per Second per Square Mile.	
Driest	Oct.	3,190	.44	July	2,600	.36	Aug.	1,210	.17	.36
2d driest	Sept.	4,800	.70	Sept.	4,320	.62	June	2,280	.33	.41
3d "	June	7,800	1.12	Oct.	6,170	.85	Oct.	2,950	.41	.55
4th "	Nov.	8,880	1.27	June	9,870	1.41	Nov.	4,500	.64	.74
5th "	Jan.	12,400	1.72	Apr.	12,180	1.74	Sept.	4,620	.66	.92
6th "	Apr.	14,700	2.10	Nov	14,280	2.04	Dec.	5,120	.71	1.30
7th "	July	16,300	2.26	Aug.	17,210	2.38	May	5,360	.74	1.41
8th "	Dec.	16,610	2.30	Mar.	18,630	2.58	July	5,700	.79	1.74
9th "	May	16,650	2.31	Dec.	21,580	2.99	Feb.	16 970	2.60	2.18
10th "	Aug.	18,660	2 58	Jan.	21,640	3.00	Jan.	22.940	3.18	2.73
11th "	Feb.	17,920	2.75	May	27,930	3.87	Apr.	22,920	3.28	3.41
Wettest	Mar.	21,140	2.93	Feb.	28,370	4.35	Mar.	43,750	6.06	4.28
Total for year	.....	159,110	.....	.....	184,780	.....	.....	138,320	.....	....
Average per square mile	.....	.....	1.87	.....	.....	2.17	.....	.....	1.63	1.66
Total for 6 dry months	182	51,830	....	182	49,420	....	183	20,680	....	....
Average per square mile, 6 dry months	.....	.....	1.22	.....	.....	1.17	.....	.....	.49	.68

TABLE NO. 33.

Average Computed Natural Flow of Croton River at Site of New Croton Dam for 32 Years.

MONTHS IN CALENDAR ORDER.

	Total Flow Mil. Gal.	Average Monthly Flow.	Average Daily Flow.	Order of Dryness.
January	541,170	16,911	545	9
February	600,920	18,778	665	11
March	785,770	24,555	792	12
April	597,780	18,681	623	10
May	370,320	11,573	373	7
June	174,600	5,456	182	3
July	121,230	3,788	122	1
August	198,410	6,200	200	5
September	169,380	5,293	173	2
October	189,750	5,930	191	4
November	346,710	10,835	361	6
December	428,860	13,402	433	8
32 years	4,524,900	Yearly average 141,401	387	

TABLE NO. 34.

Table Showing the Number of Times that each Month has Stood in each "Order of Dryness" given in Table No. 32 of Flow of Croton River for 32 Years.

Driest.	2nd.	3rd.	4th.	5th.	6th.	7th.	8th.	9th.	10th.	11th.	Wettest.
Sep. 10	Aug. 7	Oct. 7	July 5	June 8	Dec. 8	Nov. 5	May 7	Apr. 8	Jan. 8	Feb. 7	Mar. 10
July 9	Sep. 8	July 6	Oct. 6	Oct. 5	Nov. 8	May 8	Dec. 5	Dec. 5	Dec. 5	Mar. 9	Feb. 8
Aug. 6	July 7	June 6	June 8	May 4	Jan. 4	Jan. 3	Jan. 5	May 5	Feb. 5	Apr. 7	Apr. 5
Oct. 4	June 3	Aug. 5	Aug. 6	Sep. 3	Apr. 3	Feb. 3	Apr. 3	Feb. 5	Mar. 5	Nov. 3	Jan. 4
June 2	Oct. 3	Sep. 3	Sept. 2	Nov. 3	May 2	July 3	Oct. 3	Mar. 4	Apr. 3	May 2	Nov. 2
Feb. 1	Nov. 3	Jan. 2	Nov. 2	Jan. 2	June 2	June 2	Feb. 2	Jan. 3	Aug. 2	Jan. 1	Dec. 2
.....	May 1	May 1	Apr. 1	Dec. 2	Sep. 2	Aug. 2	Mar. 2	Aug. 1	Nov. 2	Aug. 1	May 1
.....	.....	Nov. 1	May 1	Feb. 1	Oct. 2	Dec. 2	Nov. 2	Nov. 1	Sep. 1	Sep. 1	.....
.....	.....	Dec. 1	Dec. 1	Mar. 1	Aug. 1	Apr. 1	June 1	.....	Oct. 1	Dec. 1	.....
.....	.....	.....	.....	Apr. 1	.....	Sep. 1	July 1	.....	.....	.....	.....
.....	.....	.....	.....	July 1	.....	Mar. 1	Sep. 1	.....	.....	.....	.....
.....	.....	.....	.....	Aug. 1	.....	Oct. 1	.....	.....	.....	.....	.....

RECORDS OF RAINFALL IN CROTON WATERSHED.

These records are of value as a guide to deducing the flow of other streams from the known flow of the Croton measured as described in the foregoing pages—according to the assumption that, other things being equal, the yearly run-off will be proportional to the yearly rainfall.

Rainfall records in almost any locality are very much more easily obtainable than records of long continued daily gaugings of a stream, indeed all of the noteworthy streams in the Atlantic States that have been accurately gauged for a term of years will hardly reach a score. Occasional infrequent gaugings of a stream are not nearly so reliable a guide to the flow, for example, of the Housatonic, the Ten Mile or the Esopus streams, as an estimate based upon such information as can be obtained of rainfall and an assumption that the percentage of this rainfall which runs off in the stream will bear the same relation as found upon the Croton, the Sudbury, the Connecticut, Merrimack, or some one of the few other streams that has been accurately gauged for a long term of years.

This rainfall record is of special value in the present study as a means of utilizing the excellent data of rainfall and run-off obtained on the Sudbury Watershed as an outside, independent means of confirming or criticising the foregoing estimate of the number of million gallons per day that the Croton Watershed can yield.

We now find, for example, that the mean annual precipitation on the Croton Watershed for the thirty-two years, 1868 to 1899, has been..... 47.83 inches.
And that for the twenty-five years, 1875 to 1899, it averaged. 48.44 inches.
During the same twenty-five years, 1875 to 1899, the rainfall on the Sudbury Watershed supplying Boston averaged. 45.83 inches.
And we therefore, knowing nothing about the measured run-off from the Croton, would infer, since Croton rainfall averages 5.7 per cent. more than on the Sudbury, that the Croton river flow, other things being equal, would be 5.7 per cent. greater.

And we find further that the per cent. of run-off averaged 49.05 per cent. on the Sudbury against 47.15 per cent. on the Croton.*

The rainfall record commonly used heretofore as giving the average for the entire Croton Watershed is that observed at Boyd's Corners. This gauge

* I had hoped to present a similar comparison with the gaugings of certain Pennsylvania streams that have been gauged accurately now for fifteen years by the Philadelphia Water Department, but although my assistant, Mr. Knowles, has completed a part of the computations, it is impossible to finish them before the time fixed for the handing in of this report.

is understood to have been set under the supervision of J. J. R. Croes, C. E., and was, so far as can be learned, the first really reliable gauge set on the Croton Watershed. This gauge itself is a good one, and well set, as would be expected, but it will be noted from the topographical map that this location is off in the northwest corner of the watershed and further from the sea coast than the middle of the watershed, and that the ground here is at an elevation above the sea higher than the mean of the land surface.

No good reason appearing why the Middle Branch Rain-gauge Records should not be used, and noting that the Carmel gauge was used by the Department of Agriculture, I attempted such brief investigation as the time afforded, into the location and character of each gauge, and into finding which of the several values published at different times for the same gauge in the same month was most probably correct.

My assistant, George W. Taber, C. E., therefore visited each rain-gauge, measured it and its height above the ground, and noted its surroundings, as per the figures 50 to 53, annexed, and investigated such discrepancies as had been noted, by a comparison with the original records, and later, Mr. Thomas Manning, keeper of the Boyd's, Carmel and Middle Branch reservoirs, very kindly took my sheet of comparisons and verified it month by month with the original records in his possession, correcting many minor errors. The records of rainfall at Old Croton Dam were similarly compared with such of the original record books as can now be found. The corrected sheets were then further reviewed and certain discordant or incrédible records, found chiefly in the early Croton records, were rejected.

Question then came as to what weight should be given to the record of each gauge. The Middle Branch and Carmel gauges are found to be of a good type, are apparently carefully and intelligently observed and are each apparently equal in accuracy to the gauge at Boyd's Corners.

The location of the Middle Branch gauge is the most central to the watershed of any of these four rain-gauges.

Taking an outline map of the Croton Watershed showing the location of the gauges, and for the moment disregarding the elevation above sea level, we find, by marking off the map into divisions, that the percentage of the entire area that may be considered represented by each rain-gauge is, in round numbers:

Boyd's	about	15	per cent.
Carmel	"	20	"
Middle Branch	"	30	"
Croton	"	35	"

When comparison is made of altitude of the gauge locations above sea level with that of the general surface of the entire watershed, the Croton gauge is found to be much lower than the average, and Boyd's a little higher. The Croton gauge is of an old-fashioned pattern that should have been replaced years ago, and the Croton records make an unfavorable showing in care of observation, particularly the early records, which contain many incredible entries. In later years the quality of observation at Croton is apparently much better than in the early years.

RAIN-GAUGES IN CROTON WATERSHED

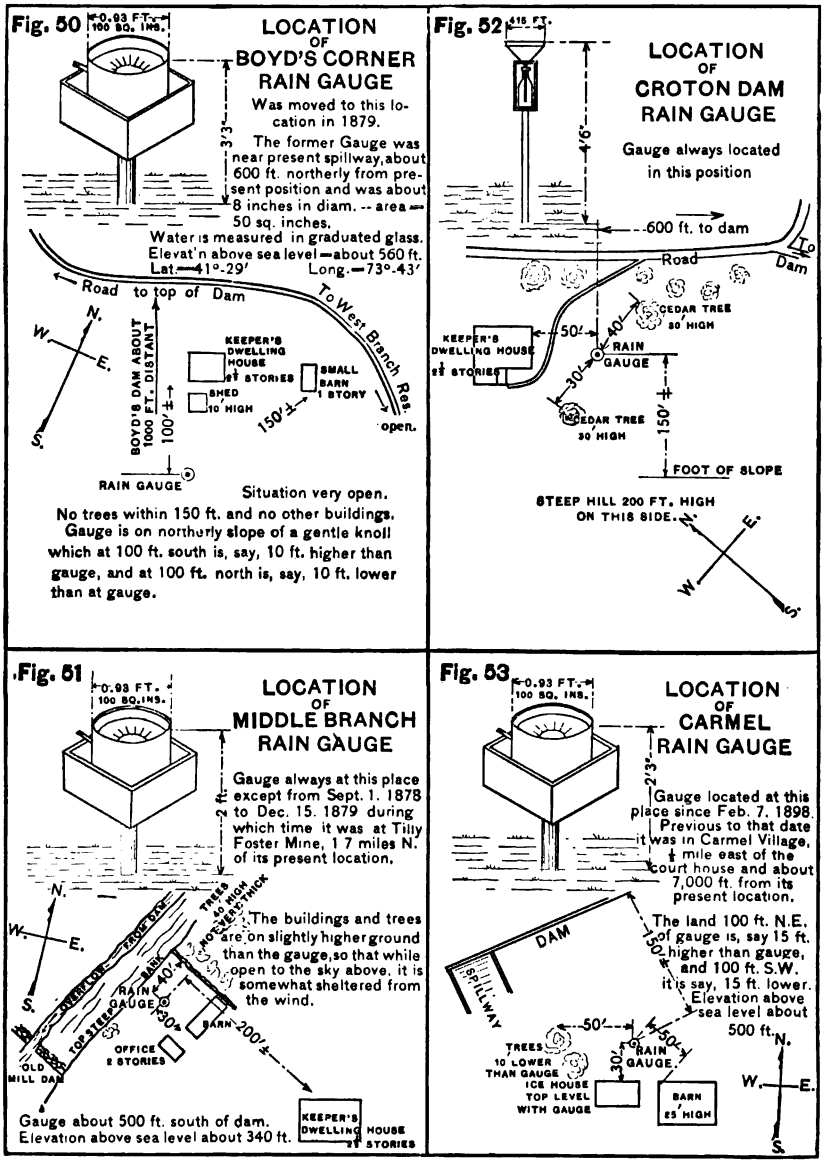


Fig. 50
0.93 FT. 100 SQ. INS.
LOCATION OF BOYD'S CORNER RAIN GAUGE
Was moved to this location in 1879.
The former Gauge was near present spillway, about 600 ft. northerly from present position and was about 8 inches in diam. -- area = 50 sq. inches.
Water is measured in graduated glass.
Elevat'n above sea level = about 560 ft.
Lat. = 41°-29'
Long. = 73°-43'
3'3"
Road to top of Dam
To West Branch Res.
BOYD'S DAM ABOUT 1000 FT. DISTANT
100' ±
KEEPER'S DWELLING HOUSE 2½ STORIES
SMALL BARN 1 STORY
SHED 10' HIGH
150' ±
open.
N.
W.
E.
S.
RAIN GAUGE
Situation very open.
No trees within 150 ft. and no other buildings.
Gauge is on northerly slope of a gentle knoll which at 100 ft. south is, say, 10 ft. higher than gauge, and at 100 ft. north is, say, 10 ft. lower than at gauge.
Fig. 52
LOCATION OF CROTON DAM RAIN GAUGE
Gauge always located in this position
4'6"
600 ft. to dam
Road
Dam
CEDAR TREE 30' HIGH
KEEPER'S DWELLING HOUSE 2½ STORIES
50'
40'
RAIN GAUGE
30'
CEDAR TREE 30' HIGH
150' ±
FOOT OF SLOPE
STEEP HILL 200 FT. HIGH ON THIS SIDE.
N.
E.
W.
S.
Fig. 51
0.93 FT. 100 SQ. INS.
LOCATION OF MIDDLE BRANCH RAIN GAUGE
Gauge always at this place except from Sept. 1. 1878 to Dec. 15. 1879 during which time it was at Tilly Foster Mine, 1 7 miles N. of its present location.
2 ft.
N.
W.
E.
S.
OVERFLOW
FENCE
TREES 40' HIGH NOT WEST THICK
BANK
40' ±
RAIN GAUGE
30'
BARN
200' ±
TOP STEPS
OFFICE 2 STORIES
OLD MILL DAM
The buildings and trees are on slightly higher ground than the gauge, so that while open to the sky above, it is somewhat sheltered from the wind.
Gauge about 500 ft. south of dam.
Elevation above sea level about 340 ft.
KEEPER'S DWELLING HOUSE 2½ STORIES
Fig. 53
0.93 FT. 100 SQ. INS.
LOCATION OF CARMEL RAIN GAUGE
2'3"
Gauge located at this place since Feb. 7. 1898. Previous to that date it was in Carmel Village, ¼ mile east of the court house and about 7,000 ft. from its present location.
The land 100 ft. N.E. of gauge is, say 15 ft. higher than gauge, and 100 ft. S.W. it is say, 15 ft. lower. Elevation above sea level about 500 ft.
DAM
SPILLWAY
150' ±
50'
150' ±
TREES 10 LOWER THAN GAUGE
ICE HOUSE TOP LEVEL WITH GAUGE
30'
RAIN GAUGE
BARN 25' HIGH
N.
W.
E.
S.

As a matter of quality of instrument, precision of observation and of elevation above sea, it appears fair to give the Croton gauge only half weight in comparison with Carmel or Middle Branch, but the proportion of drainage area to which it is the nearest, gives it greater importance; so the final adjustment of relative weight was as follows:

	By Proportion of Territory.	By Weight of Observation.	Product.	Per Cent. of Whole.	Weight of Observation Used.
Boyd's	15	1	15	21	2
Carmel	20	1	20	24	2
Middle Branch	30	1	30	37	4
Croton	35	0.5	17	18	2
	100	3.5	82	100	10

The records at the several stations were finally averaged on the following basis:

1868–1876, inclusive—	Boyd's Corner used alone.	Croton entirely rejected.
1877—	Boyd's given weight, 2.	Croton given weight, 1.
1878–1881, inclusive—	Boyd's given weight, 3.	Middle Branch weight, 5.
	Croton given weight, 2.	
1882–1899, inclusive—	Boyd's given weight, 2.	Middle Branch weight, 4.
	Croton given weight, 2.	Carmel weight, 2.

It is stated by Mr. Manning, the keeper in charge of Boyd's, Carmel and Middle Branch, that this region is subject to heavy local showers of limited area, and he cites the storm of July 2, 1897, as within his knowledge, giving 5.03 inches rainfall at Carmel and so little at two miles away that the Water Department employees working out of doors did not seek shelter. There are, however, many records of heavy rains and differences between gauges in the early days which cannot be credited. The following explanations are due for omitting certain of the records, and for including others which are extraordinary:

Croton, from 1868 to 1876, inclusive—contains so many anomalous rainfall records and so many cases where record is open to a suspicion of the snowfall not having been melted and included, that for years 1868 to 1876 the Boyd's Corner rainfalls alone are used.

Croton, July, 1877—The record of 6.48 appears large when compared to 4.65 inches for Boyd's. A storm on the 30th—4.00 inches. The Tarrytown record for July 30—3.27 inches and total for month—6.12 inches. Record let stand.

Croton, Oct., 1877—The record of 10.69 appears large when compared to 8.38 inches for Boyd's. A storm of the 4th—6.00 inches. The Tarrytown record shows 4.38 inches for this storm and 8.52 inches for whole month. Water in reservoir rose 6½ feet in 5 days. Record let stand.

Croton, Sept., 1878—Excess apparently comes from rainfall recorded as 10.40 inches on September 4 and 5, water in river suddenly rose to 44 inches over dam. On day previous a record of 4.60 inches rainfall was made with river remaining below crest of dam 27 hours later. This is incredible. Record for September excluded.

Croton, Dec., 1878—Excess apparently comes mostly in remarkably heavy rain of December 11=7.82 inches. River rose to 74 inches over dam, one of the greatest floods known. Tarrytown for same showed only 3.69 inches. Record let stand.

Croton, Aug., 1879—Excess apparently comes mainly in storm of Aug. 17 to 19—5.50 inches. This is confirmed by Tarrytown—4.63. So record is let stand.

Croton, Jan., 1881—Storm of Jan. 10, entered as 5.61 inches in Keeper's record, apparently a mistake. Snowfall was 5 inches. Temperature rose to 33 degs. F., probably total of rain and melted snow was 0.61. Monthly total therefore taken as 5.32 instead of 10.32, which has also been done in D. P. W. report.

Croton, March, 1881—Discrepancy March and April, due perhaps to entering rainfall which occurred on March 31, a day later in the other record. Record included.

Croton, Jan., 1882—Record of 8.92 inches appears large when compared to 4.49 inches at Boyd's, and 4.69 inches at Mid. Br. Tarrytown record—6.30 inches, and all rains are less that at Croton. Storm Jan. 21—2.11 inches is evidently an error, as water only ran from 7 inches to 10 inches over dam. Exclude Croton this month.

Croton, Feb., 1883—Record of Feb., 1883—7.11 inches appears large when compared to Boyd's—5.21 inches, and Mid. Br.—4.79. Tarrytown—3.15 inches and 13 inches of snow—4.15 —Storm of 14th and 15th, Croton—2.76. Tarrytown—0.70 inch. Exclude Croton in this month.

Croton, July, 1883—5.80 inches appears large when compared to 4.29 inches at Boyd's, 4.22 inches at Mid. Br. But Carmel—5.92. Tarrytown—3.30. Let Croton record stand.

Croton, Jan., 1884—9.01 appears large when compared to Boyd's 5.15, Mid. Br. 4.78 and Carmel 4.76. Storm on Jan. 8—4.97 inches. Tarrytown record for same storm—2.20 inches, and total for month—5.37 inches. Croton excluded.

Croton, Feb., 1884—8.37 appears large. Storm of 26-28—3.27 inches. Tarrytown same storm—1.16 inches, and total—5.46 inches. Exclude Croton.

Croton, Feb., 1885—9.19 appears large. Storm of 16th—4.85 inches and Tarrytown record—2.24 inches, with total of 4.70 inches. Croton excluded.

Croton, Jan., 1886—7.79 inches appears large. Storm of Jan. 5—3.79 inches. Tarrytown record for same storm—2.25, and total for month—4.78 inches. Exclude Croton.

It has been noticed that during months when heavy storms occur, the Croton record is larger than any other, and that an individual storm will commonly have a larger record at Croton than at Tarrytown.

The high percentage of run-off for 1868 is remarkable. The excluded Croton rainfall record for that year gave 88 inches—a sub-tropical figure which is incredible; but it may be that while Croton was too high, Boyd's was too low for a fair average over whole watershed.

TABLE NO. 35.

Rainfall on Croton Watershed.

Total inches depth of rain and melted snow collected during month.

	1868.			1869.			1870.		
	Boyd's.	Croton.	Value Used.	Boyd's.	Croton.	Value Used.	Boyd's.	Croton.	Value Used.
January	2.90	3.23	2.90	3.79	5.40	3.79	4.51	3.70	4.51
February	1.38	1.52	1.38	3.64	5.75	3.64	6.40	6.37	6.40
March	2.55	3.91	2.55	5.48	9.51	5.48	3.80	7.23	3.80
April	3.87	5.47	3.87	2.11	3.38	2.11	5.45	4.95	5.45
May	8.79	13.78	8.79	4.52	6.72	4.52	2.30	2.71	2.30
June	4.53	7.11	4.53	3.59	1.19	3.59	2.06	3.50	2.06
July	2.13	3.65	2.13	2.26	2.06	2.26	3.43	2.75	3.43
August	6.98	13.05	6.98	1.92	1.97	1.92	5.10	7.71	5.10
September	9.33	20.47	9.33	3.20	2.64	3.20	2.85	2.36	2.85
October	0.87	0.63	0.87	9.46	8.93	9.46	4.73	7.62	4.73
November	4.65	7.14	4.65	2.43	7.23	2.43	2.51	3.74	2.51
December	2.35	2.50	2.35	5.96	5.74	5.96	1.49	1.20	1.49
Total	50.33	82.46	50.33	48.36	60.52	48.36	44.63	53.84	44.63

	1871.			1872.			1873.		
	Boyd's.	Croton.	Value Used.	Boyd's.	Croton.	Value Used.	Boyd's.	Croton.	Value Used.
January	3.80	1.18	3.80	1.44	0.76	1.44	5.66	2.96	5.66
February	3.81	0.12	3.81	1.22	1.29	1.22	3.09	1.40	3.09
March	4.27	5.62	4.27	2.59	3.57	2.59	3.08	1.90	3.08
April	3.01	4.92	3.01	3.04	0.70	3.04	3.77	3.17	3.77
May	3.45	5.74	3.45	3.69	3.93	3.69	2.91	3.02	2.91
June	5.73	8.62	5.73	4.00	3.65	4.00	0.71	0.14	0.71
July	5.07	5.33	5.07	4.34	5.11	4.34	2.21	4.44	2.21
August	5.24	9.48	5.24	5.99	7.83	5.99	5.73	9.91	5.73
September	1.44	1.47	1.44	3.69	3.17	3.69	3.73	5.36	3.73
October	6.18	7.89	6.18	2.15	1.80	2.15	5.13	4.85	5.13
November	4.35	7.71	4.35	4.91	4.51	4.91	3.72	2.16	3.72
December	2.59	0.42	2.59	3.68	1.80	3.68	4.13	2.37	4.13
Total	48.94	58.50	48.94	40.74	38.12	40.74	43.87	41.68	43.87

Rainfall on Croton Watershed—Continued.

	1874.			1875.			1876.		
	Boyd's.	Croton.	Used.	Boyd's.	Croton.	Used.	Boyd's.	Croton.	Used.
January	6.96	5.98	6.96	2.74	2.01	2.74	1.42	1.68	1.42
February	2.78	0.17	2.78	3.47	3.83	3.47	4.01	7.92	4.91
March	1.57	0.54	1.57	4.99	5.87	4.99	6.33	14.64	6.33
April	6.31	3.49	6.31	3.04	3.78	3.04	4.43	3.82	4.43
May	1.99	1.59	1.99	1.08	1.36	1.08	3.99	3.87	3.99
June	3.57	2.26	3.57	3.02	2.78	3.02	2.52	4.34	2.52
July	5.98	5.96	5.98	3.10	7.34	3.10	3.42	6.03	3.42
August	2.75	4.22	2.75	10.33	13.06	10.33	1.20	2.51	1.20
September	3.56	4.32	3.56	2.11	1.76	2.11	5.21	4.37	5.21
October	2.40	1.90	2.40	3.61	4 27	3.61	1.50	2.13	1.50
November	2.72	2.68	2.72	4.61	6.00	4.61	3.40	3.39	3.40
December	1.78	0.99	1.78	1.56	1.86	1.56	2.35	0.50	2.35
Total	42.37	34.10	42.37	43.65	53.92	43.66	40.68	61.20	40.68

	1877.			1878.			
	Boyd's.	Croton.	Average.	Boyd's.	Croton.	Middle Branch.	Average.
January	2.68	3.22	2.86	4.49	4.30	5.03	4.71
February	0.80	1.21	0 94	3.65	5.90	3.56	4 06
March	7.66	8.98	8.11	3.10	2.87	3.07	3.04
April	2.35	2.72	2.47	2.85	3.00	3.18	3.05
May	0.85	0.57	0.76	4.97	4.11	4.72	4.67
June	4.95	5.58	5.16	4.65	3.13	4.33	4.18
July	4.65	6.48	5.26	4.28	3.86	3.98	4.05
August	2.54	3.18	2.75	2.06	2.63	4.55	3.60
September	1.49	1.09	1.36	6.61	(15.86)	8.56	7.83
October	8.38	10.69	9.15	3.78	6.07	3.51	4.10
November	8.16	7.54	7.95	4.36	6.27	4.46	4.79
December	1.52	1.35	1.46	8.74	(13.28)	6.94	7.62
Total	46.03	52.61	48.23	54.14	(71.28)	55.89	55.70

Rainfall on Croton Watershed—Continued.

	1879.				1880.			
	Boyd's.	Croton.	Middle Branch.	Average.	Boyd's.	Croton.	Middle Branch.	Average.
January	2.52	4.56	2.24	2.79	3.73	2.96	3.44	3.43
February	2.85	4.98	2.57	3.14	3.32	3.33	3.47	3.40
March	4.96	5.76	3.86	4.57	3.79	4.59	3.70	3.90
April	5.10	4.17	4.80	4.76	3.99	3.28	3.44	3.57
May	2.45	2.74	2.08	2.29	1.07	1.10	1.00	1.04
June	5.53	4.81	5.29	5.27	1.25	1.47	1.46	1.40
July	5.95	5.15	6.12	5.87	5.79	6.56	5.62	5.86
August	5.83	9.29	6.69	6.95	3.60	5.25	4.05	4.16
September	3.43	2.88	3.42	3.32	2.69	2.64	2.16	2.42
October	0.95	0.60	0.57	0.69	3.25	2.86	2.56	2.83
November	3.47	2.99	2.63	2.95	2.97	2.54	1.85	2.32
December	4.26	5.64	4.06	4.44	2.49	3.16	2.44	2.59
Total	47.30	53.57	44.27	47.04	37.94	39.74	35.19	36.92

	1881.					1882.				
	Boyd's.	Croton.	Middle Branch.	Carmel.	Average.	Boyd's.	Croton.	Middle Branch.	Carmel.	Average.
January	4.19	5.32	5.06	....	4.85	4.49	(8.92)	4.69	4.94	4.68
February	5.98	6.70	4.65	....	5.25	5.97	5.27	5.94	5.48	5.72
March	6.14	9.73	5.50	....	6.54	4.92	3.71	3.75	3.80	3.59
April	1.67	0.86	1.20	....	1.27	1.42	1.46	1.31	1.58	1.42
May	5.11	2.74	3.89	....	4.03	6.30	6.44	5.51	5.64	5.92
June	4.35	5.27	4.02	....	4.67	3.04	2.57	2.33	3.44	2.74
July	2.66	1.82	2.63	....	2.48	3.66	3.19	2.77	3.24	3.13
August	1.66	2.75	2.82	....	2.46	3.92	3.82	2.35	3.35	3.16
September	0.75	0.41	0.94	....	0.78	14.41	16.26	13.43	15.61	14.63
October	3.75	1.76	2.94	....	2.95	3.36	2.77	2.79	2.53	2.86
November	4.50	5.60	5.53	....	5.23	1.66	1.71	1.49	1.72	1.61
December	6.02	7.54	5.92	5.52	6.18	2.68	3.17	2.15	2.32	2.49
Total	46.08	50.50	45.70	5.52	46.69	55.83	(59.29)	48.51	53.87	52.35

Rainfall on Croton Watershed—Continued.

	1883.					1884.				
	Boyd's.	Croton.	Middle Branch.	Carmel.	Average.	Boyd's.	Croton.	Middle Branch.	Carmel.	Average.
January	2.98	5.27	2.88	3.00	3.40	5.15	(0.01)	4.78	4.76	4.87
February	5.21	7.11	4.79	4.98	5.38	6.23	(8.37)	5.27	5.30	5.52
March	1.87	2.22	1.56	1.68	1.78	4.82	3.31	3.77	4.25	3.99
April	3.91	3.94	2.89	3.44	3.42	2.96	3.15	2.73	2.82	2.88
May	2.86	2.45	2.39	2.72	2.56	4.33	3.92	3.70	3.90	3.90
June	5.63	3.83	3.50	6.11	4.52	1.95	3.30	2.54	2.27	2.52
July	4 29	5.80	4.22	5.92	4.89	6.86	5.56	6.43	6.26	6.31
August	2.10	2.55	3.08	2.65	2.69	5.38	8.87	7.36	5.49	6.89
September	2.52	2.47	2.80	2.48	2.61	0.96	0.34	0.95	1.07	0.85
October	7.03	5.53	6.06	6.50	6.24	3.65	1.32	3.23	3.18	2.92
November	1.72	1.13	1.73	1.51	1.56	4.37	5.23	3.86	3.88	4.24
December	3.45	4.84	3.32	3.33	3.65	7.34	6.58	5.93	6.17	6.39
Total	43.62	47.14	39.22	44.32	42.70	54.00	(58.96)	50.55	49.35	51.28

	1885.					1886.				
	Boyd's.	Croton.	Middle Branch.	Carmel.	Average.	Boyd's.	Croton.	Middle Branch.	Carmel.	Average.
January	5.59	6.18	5.02	5.23	5.41	5.99	(7.79)	5.23	5.24	5.42
February	4.66	(9.19)	4.44	4.24	4.44	4.87	6.52	4.22	4.77	4.92
March	1.26	1.62	1.14	1.12	1.26	3.86	4.48	3.86	3.48	3.91
April	2.09	2.51	2.12	2.49	2.27	3.52	5.09	3.65	3.29	3.84
May	2.44	1.87	1.85	2.46	2.09	4.54	4.02	4.59	3.68	4.28
June	1.19	0.99	1.18	1.31	1.17	3.39	3.35	3.08	3.49	3.28
July	5.26	4.87	4.95	5.04	5.01	4.76	4.46	5.68	5.97	5.31
August	7.35	7.33	6.52	5.82	6.71	3.21	3.58	2.92	2.82	3.09
September	1.09	0 87	1.19	1.16	1.10	2.30	1.43	2.69	2.35	2.29
October	5.13	5.96	4.70	5.36	5.17	2.28	2.51	1.95	1.99	2.14
November	5.99	6.43	5.76	5.87	5.96	5.57	5.13	5.29	5.66	5.39
December	3.78	3.38	2.41	3.42	3.08	4.29	3.19	4.02	3.85	3.87
Total	45.83	(51.50)	41.28	43.52	43.67	48.58	(51.55)	47.18	46.59	47.74

Rainfall on Croton Watershed—Continued.

	1887.					1888.				
	Boyd's	Croton.	Middle Branch.	Carmel.	Average.	Boyd's.	Croton.	Middle Branch.	Carmel.	Average.
January	5.68	6.44	5.65	5.44	5.77	5.56	4.01	5.24	5.58	5.31
February	5.95	7.08	6.30	5.81	6.29	5.07	4.63	5.15	5.11	5.02
March	3.60	3.59	3.73	3.96	3.72	6.44	3.32	6.69	5.89	5.81
April	3.47	3.02	2.90	3.17	3.09	2.68	2.46	2.53	2.67	2.57
May	0.32	0.18	0.30	0.47	0.31	6.17	6.73	5.77	5.76	6.04
June	7.70	6.80	6.25	6.93	6.79	2.19	2.76	1.97	2.33	2.24
July	13.55	10.72	11.12	9.64	11.23	2.24	2.80	2.40	2.22	2.41
August	6.85	4.37	7.56	7.30	6.73	6.87	6.49	6.58	6.49	6.60
September	1.90	1.47	1.81	1.51	1.70	10.77	8.19	10.05	10.92	10.00
October	3.12	3.77	3.79	3.32	3.56	4.80	4.47	4.89	4.39	4.69
November	2.69	2.17	2.61	2.52	2.52	4.49	3.33	4.20	4.12	4.07
December	6.71	4.99	5.09	6.22	5.58	6.13	5.59	5.89	6.16	5.93
Total	61.54	54.60	57.11	56.11	57.29	63.41	55.68	61.36	61.64	60.69

	1889.					1890.				
	Boyd's.	Croton.	Middle Branch.	Carmel.	Average.	Boyd's.	Croton.	Middle Branch.	Carmel.	Average
January	5 14	5.15	4.84	5.28	5.05	1.97	2.51	2.73	2.76	2.54
February	2.33	2.40	2.22	2.08	2.25	4.94	3.30	4.55	4.56	4.38
March	1.86	1.79	1.54	2.09	1.76	5.66	7.19	6.01	5.59	6.09
April	4.42	5.48	4.23	3.91	4.45	3.03	3.71	3.72	2.96	3.43
May	3.22	2.21	3.17	3.04	2.96	5.74	5.57	5.92	6.06	5.84
June	4.76	3.18	3.67	4.06	3.87	3.56	3.75	4.63	3.26	3.97
July	7.49	8.39	11.71	9.52	9.77	5.46	4.70	5.08	5.05	5.07
August	2.90	5.21	3.43	3.69	3.73	4.70	4.41	3 91	4.44	4.27
September	6.13	5.67	7.14	6.03	6.42	6.86	4.58	7.32	7.80	6.78
October	5.09	3.39	3.35	4.19	3.87	7.63	6.29	6.49	6.76	6.73
November	8.01	8.44	8.73	8.74	8.54	1.12	0.81	1.09	1.33	1.09
December	2.94	2.92	3.25	2.79	3.03	3.69	2.78	4.41	3.99	3.86
Total	54.29	54.23	57.28	55.42	55.70	54.36	49.60	55.86	54.56	54.05

Rainfall on Croton Watershed—Continued.

	1891.					1892.				
	Boyd's.	Croton.	Middle Branch.	Carmel	Average.	Boyd's.	Croton.	Middle Branch.	Carmel.	Average.
January	9.76	8.11	8.88	9.71	9.06	5.95	5.96	6.43	6.12	6.18
February	6.00	4.58	6.00	5.91	5.71	1.22	1.28	1.07	1.05	1.14
March	3.36	3.53	3.42	3.39	3.42	3.92	2.92	3.71	3.81	3.61
April	3.77	2.66	2.76	3.14	3.02	1.08	1.28	1.04	0.95	1.08
May	1.36	2.30	1.49	1.30	1.59	5.74	4.99	5.84	5.41	5.56
June	1.81	2.42	2.30	1.83	2.13	3.84	4.28	3.19	3.22	3.54
July	3.03	4.59	4.57	3.65	4.08	5.05	4.92	6.06	6.46	5.71
August	5.61	4.21	5.31	4.80	5.05	6.13	5.19	5.93	7.41	6.12
September	1.87	2.31	2.00	1.67	1.97	2.65	2.18	2.04	2.33	2.25
October	2.21	2.24	2.47	2.23	2.32	0.92	0.77	0.94	1.10	0.93
November	3.86	3.31	3.36	3.57	3.49	7.85	6.46	6.93	7.37	7.12
December	5.65	4.99	5.51	5.14	5.36	1.15	1.16	1.00	0.79	1.04
Total	48.29	45.25	48.07	46.34	47.20	45.50	41.39	44.24	46.02	44.28

	1893.					1894.				
	Boyd's.	Croton.	Middle Branch.	Carmel.	Average.	Boyd's.	Croton.	Middle Branch.	Carmel.	Average.
January	4.03	3.93	3.21	3.64	3.61	3.40	2.48	3.33	3.57	3.22
February	8.05	5.78	7.86	7.59	7.43	4.99	4.23	4.54	4.68	4.60
March	4.52	3.85	4.37	4.12	4.25	1.62	1.34	2.02	1.84	1.77
April	3.55	2.99	3.02	3.08	3.13	3.11	2.23	2.84	3.14	2.83
May	8.81	6.31	7.90	8.55	7.88	6.90	4.73	5.49	5.99	5.72
June	2.44	2.94	2.08	2.36	2.38	1.59	2.20	1.51	0.98	1.56
July	2.38	3.97	2.56	2.93	2.88	1.75	1.73	3.46	3.01	2.68
August	7.06	7.36	7.35	6.55	7.13	1.45	3.01	5.80	2.53	3.71
September	2.65	3.09	2.24	2.56	2.56	7.69	6.96	6.16	6.54	6.70
October	6.42	4.54	5.32	5.33	5.39	5.99	4.33	5.97	6.32	5.72
November	3.32	2.99	3.25	2.86	3.13	4.12	5.21	4.67	4.46	4.63
December	5.34	4.56	5.11	5.38	5.10	4.43	3.77	4.30	4.16	4.19
Total	58.57	52.31	54.87	54.95	54.87	47.04	42.22	50.09	47.22	47.33

Rainfall on Croton Watershed—Continued.

	1895.					1896.				
	Boyd's.	Croton.	Middle Branch.	Carmel.	Average.	Boyd's.	Croton.	Middle Branch.	Carmel	Average.
January	4.85	3.74	4.91	5.10	4.70	1.09	0.89	1.30	1.10	1.14
February	1.85	1.67	2.10	1.57	1.86	8.41	5.94	7.18	7.09	7.27
March	1.88	2.00	2.35	2.01	2.12	8.30	9.78	6.80	6.90	7.71
April	5.63	3.97	4.42	4.44	4.57	1 48	1.05	1.13	1.20	1.20
May	2.29	1.60	2.41	1.86	2.11	3.48	3.02	2 93	2.79	3.03
June	1.94	2.05	2.82	2.12	2.35	3.47	4.65	4.00	3.32	3.89
July	3.80	6.11	4.48	4.76	4.73	3.98	6.50	4.28	3.54	4.52
August	3.10	3.55	4.72	4.45	4.11	4.60	2.81	3.74	4.32	3.84
September	1.06	3.16	0.78	0.81	1.32	6.54	4.89	5.17	5.18	5.39
October	3.55	3.58	3.74	3.60	3.64	2.21	1.93	2.39	2.63	2.31
November	2.98	4.84	5 06	4.95	4.58	3.96	3.70	3.38	3.43	3.57
December	4.85	4.72	4.12	4.62	4.49	2.72	1.21	1.93	2.10	1.98
Total	37.78	40.93	41.91	40.29	40.58	50.24	46.37	44.23	44.20	45.85

	1897.					1898.				
	Boyd's.	Croton.	Middle Branch.	Carmel.	Average.	Boyd's.	Croton.	Middle Branch.	Carmel.	Average.
January	4.07	2.90	3.53	3.45	3.49	5.89	4.82	4.60	4.97	4.98
February	3.14	2.16	2.66	2.76	2.67	5.21	4.36	4.18	4.64	4.51
March	3.68	3.03	3.49	3.34	3.40	2.64	3.66	2.89	2.43	2.90
April	2.78	3.10	3.05	3.05	3.01	3.74	3.83	3.50	3.78	3.67
May	5.56	6.52	6.03	5.96	6.02	7.37	8.60	7.89	7.13	7.77
June	3.41	2.51	3.18	3.23	3.10	1.50	1.54	1.39	1.23	1.41
July	12.51	10.99	11.15	16.64	12.49	4.78	5.57	3.52	3.80	4.24
August	5.63	5.04	5.21	4.90	5.20	7.66	9.19	13.94	12.87	11.52
September	1.74	2.00	1.79	1.74	1.82	2.17	2.17	2.23	2.38	2.24
October	2.01	1.49	0.93	0.89	1.25	4.84	5.38	4.63	4.69	4.83
November	6.31	4.49	5.72	6.22	5.69	6.81	5.80	6.34	6.15	6.29
December	6.56	3.55	4.78	5.22	4.98	3.17	2.40	3.17	3.29	3.04
Total	57.40	47.78	51.52	57.40	53.12	55.78	57.32	58.28	57.36	57.40

Rainfall on Croton Watershed—Continued.

	1899.					1900.				
	Boyd's.	Croton.	Middle Branch.	Carmel.	Average.	Boyd's.	Croton.	Middle Branch.	Carmel.	Average.
January	3.37	4.73	3.78	3.59	3.85	4.18	4.05	....	4.06	....
February	5.54	4.19	5.84	5.76	5.43	....	....	....	....	....
March	7.56	5.65	6.46	7.02	6.63	....	....	....	....	....
April	1.99	1.92	1.58	1.66	1.75	....	...	....	....	....
May	1.80	2.13	2.10	1.68	1.96	....	....	....	....	....
June	5.38	4.93	5.52	6.73	5.62	....	....	...	....	....
July	5.99	6.04	6.17	5.28	5.93	....	....	....	....	....
August	0.48	1.57	0.32	0.53	0.64	....	....	....	....	....
September	8.86	5.08	7.10	7.78	7.19	....	....	....	....	....
October	1.48	1.28	1 29	1.02	1.27	....	....	....	....	....
November	2.05	1.85	1.82	2.02	1.91	....	....	....	....	....
December	2 75	2.30	2.37	2.67	2.49	....	....	....	....	....
Total	47.25	41.67	44.35	45.74	44.67	....	....	....	....	....

TABLE

Rainfall on Croton Watershed, in Total Inches, Depth each Month and Year, 1868–1899. over the Entire Watershed. Percentage of

MONTHS.	1868.	1869.	1870.	1871.	1872	1873.	1874.
January	2.90	3.79	4.51	3.80	1.44	5.66	6.96
February	1.38	3.64	6.40	3.81	1.22	2.09	2.78
March	2.55	5.48	3.80	4.27	2.59	3.08	1.57
April	3.87	2.11	5.45	3.01	3.04	3.77	6.31
May	8.79	4.52	2.30	3.45	3.69	2.91	1.99
June	4.53	3.59	2.06	5.73	4.00	0.71	3.57
July	2.13	2.26	3.43	5.07	4.34	2.21	5.98
August	6.98	1.92	5.10	5.24	5.99	5.73	2.75
September	9.33	3.20	2.85	1.44	3.69	3.73	3.56
October	0.87	9.46	4.73	6.18	2.15	5.13	2.40
November	4.65	2.43	2.51	4.35	4.01	3.72	2.72
December	2.35	5.96	1.49	2.59	3.68	4.13	1.78
Total rainfall	50.33	48.36	44.63	48.94	40.74	43.87	42.37
Total run-off	33.33	23.61	19.20	19.46	16.92	25.02	25.10
Per cent.	66.22	48.82	43.02	39.76	41.53	57.03	59.24

MONTHS.	1884.	1885.	1886.	1887.	1888.	1889.	1890.
January	4.87	5.41	5.42	5.77	5.31	5.05	2.54
February	5.52	4.44	4.92	6.29	5.02	2.25	4.38
March	3.99	1.26	3.91	3.72	5.81	1.76	6.09
April	2.88	2.27	3.84	3.09	2.57	4.45	3.43
May	3.90	2.09	4.28	0.31	6.04	2.96	5.84
June	2.52	1.17	3.28	6.79	2.24	3.87	3.97
July	6.31	5.01	5.31	11.23	2.41	9.77	5.07
August	6.89	6.71	3.09	6.73	6.60	3.73	4.27
September	0.85	1.10	2.29	1.70	10.00	6.42	6.78
October	2.92	5.17	2.14	3.56	4.69	3.87	6.73
November	4.24	5.96	5.39	2.52	4.07	8.54	1.09
December	6.39	3.08	3.87	5.58	5.93	3.03	3.86
Total rainfall	51.28	43.67	47.74	57.29	60.69	55.70	54.05
Total run-off	24.08	17.71	20.10	26.61	35.27	31.39	25.95
Per cent.	46.96	40.55	42.10	46.45	58.12	56.36	48.01

No. 36.

Natural Flow of Croton River at Old Croton Dam in Equivalent Inches, Depth for the Year Run-off to Rainfall for each Year.

1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.
2.74	1.42	2.86	4.71	2.79	3.43	4.85	4.68	3.40
3.47	4.91	0.94	4.06	3.14	3.40	5.25	5.72	5.38
4.99	6.33	8.11	3.04	4.57	3.90	6.54	3.99	1.78
3.04	4.43	2.47	3.05	4.76	3.57	1.27	1.42	3.42
1.08	3.99	0.76	4.67	2.29	1.04	4.03	5.02	2.56
3.02	2.52	5.16	4.18	5.27	1.40	4.67	2.74	4.52
3.10	3.42	5.26	4.05	5.87	5.86	2.48	3.13	4.89
10.33	1.20	2.75	3.06	6.95	4.16	2.46	3.16	2.69
2.11	5.21	1.36	7.83	3.32	2.42	0.78	14.63	2.61
3.61	1.50	9.15	4.10	0.69	2.83	2.95	2.86	6.24
4.61	3.40	7.95	4.79	2.95	2.32	5.23	1.61	1.56
1.56	2.35	1.46	7.62	4.44	2.59	6.18	2.49	3.65
43.66	40.68	48.23	55.70	47.04	36.92	46.69	52.35	42.70
24.77	21.09	20.22	27.17	19.65	12.63	19.25	24.28	13.33
56.73	51.84	41.92	48.78	41.77	34.21	41.23	46.38	31.22

1891.	1892.	1893.	1894.	1895.	1896.	1897.	1898.	1899.	Average for 32 Years.
9.06	6.18	3.61	3.22	4.70	1.14	3.49	4.98	3.85	**4.20**
5.71	1.14	7.43	4.60	1.86	7.27	2.67	4.51	5.43	**4.13**
3.42	3.61	4.25	1.77	2.12	7.71	3.40	2.90	6.63	**4.03**
3.02	1.08	3.13	2.83	4.57	1.20	3.01	3.67	1.75	**3.18**
1.59	5.56	7.88	5.72	2.11	3.03	6.02	7.77	1.90	**3.78**
2.13	3.54	2.38	1.56	2.35	3.89	3.10	1.41	5.62	**3.36**
4.08	5.71	2.88	2.68	4.73	4.52	12.49	4.24	5.93	**4.87**
5.05	0.12	7.13	3.71	4.11	3.84	5.20	11.52	0.64	**4.87**
1.97	2.25	2.56	6.70	1.32	5.39	1.82	2.24	7.19	**4.02**
2.32	0.93	5.39	5.72	3.64	2.31	1.25	4.83	1.27	**3.80**
3.49	7.12	3.13	4.63	4.58	3.57	5.69	6.29	1.91	**4.12**
5.36	1.04	5.10	4.19	4.49	1.98	4.98	3.04	2.49	**3.71**
47.20	44.28	54.87	47.33	40.58	45.85	53.12	57.40	44.67	**48.07**
23.48	17.68	29.05	20.56	15.95	23.26	25.59	29.72	22.28	**22.93**
49.75	39.93	52.94	43.44	39.31	50.73	48.17	51.77	49.88	**47.70**

Appendix No. 2.

SAFE YIELD OF THE BRONX AND BYRAM SOURCES.

It is understood to be the opinion in the Department of Water Supply that these sources can be relied upon to furnish 20 million gallons per day in a dry year. While this may be true for a year of moderately low rainfall, I am led to believe that in a succession of dry years, such as those from 1881 to 1883, or in the still drier series of three to five years which are almost certain to come two or three times in a century, not more than 15 million at most, and more likely not more than an average delivery of 14 million gallons per 24-hour day could be furnished from these sources, although managed in the very best possible manner.

As the strength of a chain is no greater than its weakest link and bears no relation to the average strength of the several links or to the strongest links, so the strength of a source of public water supply is no greater than that of its weakest year, except so far as there is reservoir capacity to store water in wet years and carry it over for use in a series of dry years; and the size of the storage volume that may be availed of is limited by practical considerations of impairment of quality likely to occur in a reservoir too long drawn down.

My estimate that the safe yield is from 14 to 15 million gallons is based upon the following data, given in the publications of the Department of Water Supply, mainly:

Total drainage area—Bronx river supply.....	13.67	square	miles.
Byram " "	8.33	"	"
Total	22.00	"	"

Storage Capacity and Evaporating Area.

Kensico Reservoir capacity.....	1,600	million gallons, area full	250	acres.	
Rye " "	1,300	" " " "	280	"	
Byram Lake "	800	" " " "	160	"	
Total "	3,700	" " " "	690	"	

Authority for the above drainage areas and storage volumes is found in the Report of the N. Y. Department of Water Supply for 1898.

Authority for area of water surface, Wegman on The Water Supply of New York, p. 92.

There are additional water surfaces within the watershed exposed to evaporation, as follows:

Wampus Pond	about	50 acres.
Byram River and Dam	"	70 "
Total water surfaces exposed to evaporation	"	810 "

Total water surfaces, 810 acres = 1.27 sq. miles = $\frac{1.27}{22}$ = 5.8 per cent.

5.8 per cent. of entire drainage area is water surface and 94.2 per cent. is land surface. Net land surface is 22.00 — 1.27 = 20.73 square miles.

This proportion of evaporating water surface is slightly larger than that upon the Croton Watershed will be after the New Croton Lake is filled in 1902.

Not all of the 3,700 million gallons of storage estimated above can be utilized. In the great drought of 1891, before Byram was added, Kensico and Rye are understood to have been drawn to the lowest level practicable.

Kensico was then drawn from depth of..... 34 feet to depth of 11.42 feet.
Rye was then drawn from (about)..... 18.5 feet to depth of 2.46 feet.
Williamsbridge was drawn from (about).... 40 feet to depth of 32.50 feet.

The corresponding depletions were:

Rye	1,350	million	gallons.
Kensico	1,200	"	"
Williamsbridge	27	"	"
Total depletion	2,577	"	"
We are informed that Byram Lake can be drawn 13 feet, giving	550	"	"
For Byram Dam, understood to contain 180 million gallons when full, assume that 100 millions can be drawn out through tunnel and canal	100	"	"
Thus giving a total available storage	3,227	"	"

But it is to be noted that the storage in Byram Lake is not of full value, because of the watershed tributary thereto being so very small that we cannot rely upon it refilling promptly after being drawn low.

Obviously the 550 million gallons of Byram Lake storage is not available for the watershed below its outlet, and we must compute this yield in two parts; the first part being that tributary to Byram Lake and the second part including all the rest. If data were more complete this latter part should be subdivided into the area above the Byram Tunnel and the area below it.

Byram Lake Yield.

The geological survey map shows a watershed area tributary to Byram Lake of only 1.25 square miles, of which the pond itself occupies 20 per cent., or 0.25 square mile. The available storage volume of this reservoir is thus $\frac{550}{1.25} = 440$ million gallons per square mile of gross area of its tributary watershed. The water surface is 20 per cent. and the safe yield by Diagram 48 is 800,000 gallons per day on the Sudbury basis and 860,000 gallons per day on the Croton basis.

Yield Above Byram Diversion Dam.

It is to be noted that with a very rapid rainfall or a rapid melting of snow the tunnel and channel from Byram Dam to Kensico Reservoir, although reported to be 75 million gallons capacity per day, might not be able to convey all of the water from the seven square miles of land surface in the drainage area tributary thereto outside of that tributary to Byram Lake. Seventy-five million gallons per 24 hours corresponds to a run-off of 0.616 inches in depth per 24 hours from a drainage area of seven square miles; wherefore, with a rainfall of, for example, 2 inches, which is very common, if 50 per cent of this found its way into the river within 24 hours, a considerable portion would of necessity be beyond the capacity of the tunnel to convey into storage, and would therefore have to run to waste. For the moment we will neglect this waste and will combine the yield from this area with the yield tributary to Kensico.

Yield Tributary to Kensico, Including Byram Diversion.

The total available storage space exclusive of Byram Lake as per above items, aggregates 2,677 million gallons $\div$ 20.75 square miles $=$ 129 million gallons per square mile of gross watershed.

The available storage on Croton Watershed after completion of Cornell Dam will aggregate 203 million gallons per square mile. Thus the Bronx and Byram sources combined have relatively less storage than Croton will have after 1902, and indeed less than the Croton has to-day.

A small watershed is almost universally subject to greater variations than a large watershed, but in this case this need not enter the computation, because this Bronx and Byram source is used in connection with the larger watershed of the Croton.

The per cent. of water surface on this area of Bronx and Byram river, exclusive of Byram Lake, is $\frac{1.02}{20.75} = 4.92$ per cent.

From Diagram No. 48 we find that according to the Sudbury experiences the safe yield of a watershed having 129 million gallons storage per square mile and 5 per cent. water is 640,000 gallons per square mile per day × 20.75 square miles = 13.3 million gallons per day, while according to the greatest droughts found in 32 years of observations on the Croton the safe yield is 675,000 million gallons per day × 20.75 square miles = 14.0 million gallons per day.

Yield of Total Bronx and Byram Watersheds.

Adding together the foregoing results and making no deduction for the occasional waste in a severe freshet over the Byram Diversion Dam, we have—

	Million Gallons Per 24 Hours.	
	On Sudbury Basis.	On Croton Basis.
Yield of area tributary to Byram Lake	1.0	1.07
Yield of area below Byram Lake and above Kensico	13.3	14.0
Total safe yield, Bronx and Byram	14.3	15.07

Rough Check from Croton.

By simple proportion, from the Croton safe yield of 275 million gallons per day according to the gross areas of watershed, the yield would be $275 \times \frac{22 \text{ square miles}}{360 \text{ square miles}} = 16.8$ million gallons per 24 hours, but this is plainly too much if the rainfall is the same, because the storage is proportionately less on the Bronx.

Because of the loss that will sometimes occur because of waste in floods on the Byram Watershed which the tunnel cannot deliver into the Kensico storage, and in recognition of the fact that the drought of 1879–1883 was less severe on the Croton than at some other points, I consider that a continuous uniform draft of 14 million gallons per 24 hours is the greatest yield that can

be relied upon from the Bronx and Byram Watersheds in a series of years of extremely low rainfall, with their present storage capacity. By the construction of a new dam between Rye and Kensico, as proposed on page 382, it would be practicable to increase this yield to about 17 million gallons per day.

MEASUREMENT OF DELIVERY OF BRONX CONDUIT.

There is no Venturi meter, weir or other device in use for measuring the daily delivery of the Bronx conduit. So far as I can learn the only means of measurement is by noting the difference of elevation of water surface between the Kensico Reservoir and the Williamsbridge Reservoir, and computing the flow by the ordinary hydraulic formula, assuming that the gates are wide open.

Mr. Birdsall tells me he has occasionally obtained a check measurement by noting the rate at which the Williamsbridge Reservoir would refill from this Bronx conduit. The gates on this conduit are understood to have been kept wide open all, or nearly all, of the time during the past year.

From a study of the records of reservoir heights it appears that the gross head acting on the Bronx conduit pipe to produce flow has averaged from 50 to 53 feet, rarely, as on August 1, 1899, reaching 60 feet. With both reservoirs even full the fall is 50 feet. The length of this conduit, which is of ordinary 48-inch cast-iron pipe, is stated to be 15.2 miles.

There is said to be no obstruction at entrance or exit except a $\frac{1}{2}$-inch screen at inlet and a 2-inch screen at outlet.

The pipe was put in service in 1884, and we may assume that it is now in substantially the same condition as the Rosemary pipe when experimented on by Mr. Fitzgerald before cleaning, after twenty years' use.

The Rosemary Pipe, 48-inch diameter, had—

Coefficient of flow in Chezy formula, when clean..... $C = 141$.
Coefficient of flow in Chezy formula, when foul..... $C = 109$.

With same coefficients I compute that when Bronx pipe was new and both reservoirs full, fall then being just 50 feet, the delivery was probably 27.4 million gallons per 24 hours. At present the delivery is probably 22.0 million gallons per 24 hours under the same conditions; that is with a fall of 50 feet.

Appendix No. 3.

HOURLY VARIATION IN CONSUMPTION OF CROTON WATER.

Night Consumption as an Index of Waste:

This investigation was made because it afforded the best available means of estimating the proportion of New York's water supply which is actually *used* in distinction from the portion *wasted.*

The results indicate that out of 115 gallons per inhabitant per day, not more than 40 gallons are used and not less than 75 gallons wasted. It must be remembered that this is from but a single series of measurements covering only two weeks, and that during these two weeks some unknown cause may have interfered with the accuracy of the measurements, yet after carefully studying the situation, I can find so far no reasonable ground for distrusting this determination, but find many facts to confirm it.

The record of draft resulting from these measurements, plotted in the lower curve of Figures 5 and 6 and in Figure 54, is analogous to the curve registered by the Deacon meter in the Deacon system of water waste prevention, but is a more sharply defined measurement.

Having first determined the varying quantity drawn from hour to hour by any convenient method of measurement, the inference as to the proportion used and the proportion wasted is based on the broad, general experience that the real *use* of water in the hours after midnight, when most people are asleep, most factories idle and most stores and offices empty, or more especially between 2 and 4 A. M., is very small in comparison with the use during the hours of daylight; particularly so after midnight on Sunday when factory tanks and house tanks have had ample time to become replenished under the higher pressure of Sunday. In most cities the amount of water drawn from the distribution pipes between 2 and 4 A. M. may be fairly assumed to consist almost entirely of leakage and waste; this fact is the basis of all work in waste prevention with the Deacon meter which has been so remarkably successful in Liverpool and Glasgow. This assumption has been proved correct in a large American commercial city by the careful work with the Deacon meter system carried on for many years in Boston, under the supervision of Dexter Brackett, C. E., and in numerous investigations into water waste made in other cities. But it does not need meters, or tests, or records to prove what

everybody will admit; that in 19 households out of 20 no water is drawn for use between 2 and 4 A. M., and that not one-tenth as many workmen work by night as by day, that not one business office or store in one hundred is doing business at these hours. Common sense and every day experience indicate that the real rate of draft of water for *use* through faucets and stop-cocks is not one-tenth part so great from 2 to 4 A. M. as from 7 A. M. to 7 P. M.

The only questions that can cause doubt about the night consumption in New York from 2 to 4 A. M., being almost wholly waste, are :

1st. Under the present conditions existing in New York, with tall buildings, feeble water pressure and some of the pipes small, and with the consequent large use of house tanks and house pumps, may it not be that the refilling of these tanks is largely done at night?

There are two main classes of house tanks, those in domestic and those in commercial or manufacturing buildings; or they may be divided on another line—those filled by gravity and those filled by pumping. The domestic gravity house tanks in the high service district are largely drawn down by day and refilled by night, but from discussing this question with leading plumbers and from general considerations it appears probable that the most of these are refilled before midnight or soon after. The house tanks filled by pumping are mainly filled by day.

A canvass made of a large number of representative commercial, manufacturing and other buildings, and by conference with building superintendents and engineers familiar with the current practice in New York, proves conclusively that the almost universal custom is to keep these tanks nearly full by pumping during the day and to rarely, if ever, do any pumping after midnight, or before 6 A. M. Many pumps were found to be fitted with automatic regulators which started the pump whenever the tank was drawn down.

2d. May there not be in New York special uses for large quantities of water at night in special lines of industry?

A visit to certain large power stations brings evidence that their hourly draft of water after midnight is only twenty per cent. of the rate of draft by day. Among offices and commercial buildings, the main telegraph offices and the large newspaper offices were the principal large night users found, also certain breweries; but if the total use by all these establishments be added together and reduced to the per capita of population, the figure is so small as to have no weight in the present discussion. I have been unable to learn of special industries demanding any uncommonly large quantities of water by night.

Indeed, bearing in mind that by the records of the City Water Register and the statement of the Water Commissioner, the total per capita consumption for commercial and manufacturing purposes is only 24 gallons, if we assumed the night rate from 12 to 6 at the high figure of one-third the mean rate (which is surely too large), it gives only 8 gallons out of the 14 to 19, which it will be seen later that I have allowed for *use* at night.

3d. May not the habit of late hours in New York increase the draft of water for *use* late at night?

While late hours are kept by many in New York, the great mass of the people are working people who rise early and retire early, and do not draw water for use between 2 and 4 A. M.

The main uncertain element is the domestic gravity house tank, and for a possible late hour in refilling these I have allowed very liberally in calling the night possibly 19 gallons instead of 4 per capita, for it can be demonstrated that if we multiply the average size by the probable number, and then make any reasonable assumption as to the hours between which they are refilled, their possible capacity as a factor for equalizing the day and night flow will still be very small in comparison with the Central Park reservoirs.

Having thus demonstrated that the rate of consumption of water between the hours of 2 and 4 A. M. is a fair index to the waste of water from leaks in pipes and fixtures, percolation from earth reservoirs or any other constant leakage, the next question is of the proper measurement of this actual rate of consumption from hour to hour.

Method of Measurement of the Variation in Consumption of Croton Water at Different Hours of the Day.

After a careful study of possible methods of determining this consumption in Croton water in the hours after midnight by the insertion of Pitot tube velocity-meters in certain of the main pipes, or of estimating it by the indications of recording pressure-gauges set in various parts of the city, the simplest and best method by far, appeared to be that of measuring the fluctuation in the Central Park Reservoirs. All of the Croton water was at this time coming to New York through the New Croton Aqueduct. The rate of delivery of the aqueduct is constant and uniform to a very remarkable degree. This flow in the aqueduct is regulated solely by the gates at the Croton Dam, which gates are so readjusted whenever the water level in Croton Lake changes that a constant depth of flow, varying rarely one-fourth of an inch, or one-tenth of one per cent., for weeks at a time, is maintained in the aqueduct. This point where the flow is thus measured into aqueduct being thirty-three miles away from Central Park, and about 35 feet above it in elevation, the rate of flow in the aqueduct is not changed to any perceptible degree by any ordinary variation in height of water at Central Park. The rate of this delivery of the New Aqueduct in gallons per 24 hours is commonly estimated from the depth of water admitted into the aqueduct at Croton Dam, but at about the time of these experiments, a gauging of the flow in the New Aqueduct was made by Mr. F. W. Watkins, Assistant Engineer, Aqueduct Commissioners (see Table No. 17, p. 173, Appendix No. 1), which gives us the rate of delivery of the aqueduct during the experiments now under discussion with more than ordinary accuracy.

The Central Park Reservoirs serve as an equalizer between the demands of the distribution system and the delivery of the aqueduct, and commonly rise about 3 inches during the night while the draft is less, and fall a similar amount under the heavier draft of the daytime; so our method consisted

simply in measuring with the greatest practicable accuracy the rate of its rise or fall in each hour of the twenty-four.

The combined area of the three reservoirs being about 123 acres and the change of level so slight, it was feared that waves caused by the wind or other disturbances might interfere with our measurements. An earnest effort was therefore directed toward making these measurements with precision and frequency. At the large reservoir a portable hook-gauge was used, and a staging arranged so that the observer's eye could be near the water surface and a position chosen as free from wave interference as practicable. The first series of observations were from Saturday noon, December 2, to Monday noon, December 4, and were made for the purpose of learning the Sunday minimum and to get the general run of the fluctuations and to test the method. Two observers, working in 12-hour shifts, took continuous observations at 3-minute and 6-minute intervals. A station was selected inside the main gate-house at the New Reservoir, and during this preliminary test the Old Reservoir was not observed, it being assumed that connected as they were, both reservoirs would rise and fall at substantially the same rate, or that the ratio of one to the other could be subsequently determined.

This location for the hook gauge adopted in the preliminary test was open to the possible criticism of there being a slight current between the reservoir and the point of observation, which was in the large open gate-chamber supplying the suction pipes for the Ninety-eighth street high service station, but care was taken to note that the gates between this chamber and the reservoir were wide open, and that the screens were clean and that the minimum area of opening between the reservoir and this gate-chamber was 75 square feet, while the area of the two outlet pipes was only 14 square feet, and that the maximum rate of draft through these chambers and passages was about 25 million gallons per 24 hours = 38.6 cubic feet per second, which would cause a velocity of flow of only 0.52 feet per second through this minimum area. The head due this maximum velocity is only 0.004 foot, and the variations in this head must have been entirely inappreciable except for the six hours on Saturday afternoon, December 2, while pumps were stopped. Subsequently measurements and comparisons proved that observations inside this gate-chamber give heights that accurately represented the change of level in the reservoir outside of the gate-house.

In the second and more complete series of observations from which the main curve of Diagram No. 5 is drawn, and which was continued for seven

days at 6-minute intervals continuously, day and night, error in measurements of water level of the New Reservoir were guarded against by selecting four different points from which check measurements were taken; one of these points being in a stillbox specially constructed for these observations, located outside the gate-house, submerged directly in the reservoir, and consisting of a wooden box of about 1.5 square feet sectional area, communicating with the reservoir through a 1-inch hole. These check observations, taken at least once an hour, served also to relieve the monotony of continually observing the same gauge and to break up the continuance of any possible misreading.

In this second series, which ran from December 9 to December 16, inclusive, four observers worked two at a time in 12-hour shifts, and observations of the height of water in each of the two divisions of the Old Reservoir were also taken at 6-minute intervals, the measurements at the Old Reservoir being made upon scales graduated to hundredths of a foot set in stillboxes submerged one in each of the two divisions of the reservoir at a point near their common overflow. No water wasted over either overflow during any of these experiments. No ice formed on the surface of the reservoirs or in the stillboxes at any time. For about one-half of the time during these observations there was very little wind, and the opportunity was excellent for a very close measurement of the surface, but on part of the remaining time there were considerable wind oscillations, but the use of the stillbox, with the precision of the measurements possible with the gauge, is found to have given very concurrent results. The computations for the first series are not included in the following sheets, but the result is plotted upon the same diagram with the other (Fig. 5).

The results of these two sets of observations, taken a week apart, confirm each other remarkably, and the difference between the two lines Saturday afternoon, December 2, compared with Saturday afternoon, December 9, is fully accounted for by the fact that on the afternoon of December 2, the Ninety-eighth street high-service pumps were shut down. The excess on Sunday, December 3, of this line over the line for Sunday, December 10, is similarly accounted for by the refilling of the high-service reservoir then taking place, while the pumps were run at more than their ordinary speed for that time of day on Sunday.

The fluctuation of height in the high-service reservoir was not regularly observed during these experiments except throughout one day. Its area is only about one acre, whereas the Central Park Reservoirs contain about 123 acres of water surface. Therefore, any possible rise and fall, other than

from Saturday noon, December 9, to Sunday, December 10, could not have materially affected the measurements. To test this a computation was made from the 24 hours' observations on high service fluctuations mentioned above.

It may appear that the following records were hardly worth publishing in detail, but as question is likely to be raised concerning the method, it has appeared best to present the data very fully for the Central Park observations.

The Brooklyn records are not reproduced. They were worked up by substantially the same method, except that here the variable pump delivery had to be very carefully observed.

The hourly records from the Fall River and Woonsocket observations, similarly, are not reproduced, but followed very much the same methods.

TABLE NO. 37.

Hourly Variation in the Consumption of Croton Water, Shown by Rise and Fall of Reservoirs in Central Park, New York City, while Aqueduct is Delivering at a Constant Rate of Flow.

From observatons of George A. Taber, C. E., and assistants, under supervision of John R. Freeman, C. E. December, 1899.
Observations made at 6 minute intervals, first plotted on millimeter cross section paper.
Vertical scale, one millimeter = 0.002 feet. Horizontal scale, one hour = 50. millimeters.
The heights of water in reservoirs used below are mean values taken from these plotted curves.

Official estimate total population January 1, 1900, by New York City Board of Health, Manhattan	1,980,223	inhabitants.
Official estimate total population January 1, 1900, by New York City Board of Health, Bronx	168,867	"
Total	2,149,090	inhabitants.
Department of Water Supply report, 1898, p. 95, estimates supply from private water sources 190,000 gallons per day, at rate common in suburbs with scant supply, using 60 gallons per inhabitant per day, this would supply	32,000	"
Leaving to be supplied from Croton and Bronx	2,117,090	inhabitants.

Croton supply is 92 per cent. of combined Croton and Bronx supplies, so call population supplied from Croton 92 per cent. of 2,117,000 = 1,947,000 inhabitants.

Time.		New Reservoir. Water about 3 feet deep over division-wall, so both divisions at same level. At this average level 1.00 feet in depth contains 29.9 million gallons.			Old Reservoir. North Division. At this average level 1.00 feet in depth contains 5.88 million gallons.			Old Reservoir. South Division. At this average level 1.00 feet in depth contains 4.16 million gallons.						
Beginning of Interval.	Duration of Interval.	Mean gauge reading at beginning of interval, feet.	Rise or fall of surface during interval, feet.	Excess or deficiency of consumption over delivery from this reservoir in million gallons, during this hour.	Mean gauge reading at beginning of interval, feet.	Rise or fall of surface during interval, feet.	Excess or deficiency of consumption over delivery from this reservoir in million gallons during this hour.	Mean gauge reading at beginning of interval, feet.	Rise or fall of surface during interval, feet.	Excess or deficiency of consumption over delivery from this reservoir in million gallons during this hour.	Total excess or deficiency of consumption over delivery during this hour in million gallons.	Delivery by Croton Aqueduct during this hour in million gallons.	Total consumption of water from all distribution pipes during this hour in million gallons.	Rate of consumption per inhabitant per 24 hours, including both high and low service.
FRIDAY, DECEMBER 8, 1899.														
12 noon	1 hour.	3.690	—.030	—.897	3.222	—.036	—.212	3.180	—.034	—.141	—1.250	9 50	10.75	132
1 hr. 0 min	"	3.660	—.034	—1.016	3.186	—.036	—.212	3.146	—.033	—.137	—1.365	9.50	10.87	134
2 hr. 0 min	"	3.626	—.039	—1.116	3.150	—.037	—.218	3.113	—.028	—.116	—1.500	9.50	11.00	136
3 hr. 0 min	"	3.587	—.032	—.957	3.113	—.038	—.223	3.085	—.018	—.075	—1.255	9.50	10.76	133

4 hr. o min.....	"	3.555	—.030	—.897	3.075	—.030	—.176	3.067	—.045	—.187	—1.260	9.50	10.76	133
5 hr. o min.....	"	3.525	—.030	—.897	3.045	—.010	—.059	3.022	—.026	—.108	—1.064	9.46	10.52	130
6 hr. o min.....	"	3.495	—.020	—.598	3.035	—.041	—.241	2.996	—.036	—.150	—.989	9.42	10.41	128
7 hr. o min.....	"	3.475	—.015	—.448	2 994	—.006	—.035	2.960	—.006	—.025	—.508	9.37	9.88	122
8 hr. o min.....	"	3.460	+.002	+.060	2.988	—.018	—.106	2.954	—.003	—.012	—.058	9.33	9.39	116
9 hr. o min.....	"	3.462	+.006	+.179	2.970	—.004	—.024	2.951	+.003	+.012	+.167	9.33	9.16	113
10 hr. o min.....	"	3.468	+.016	+.478	2.966	+.018	+.106	2 934	+.019	+.079	+.663	9.33	8.67	107
11 hr. o min.....	"	3.484	+.026	+.777	2.984	+.002	+.012	2.973	+.025	+.104	+.893	9.33	8.44	104

SATURDAY, DECEMBER 9, 1899.

12 midnight.....	1 hour.	3.510	+.030	+.897	2.986	.000	.000	2.998	+.017	+.071	+.968	9.33	8.36	103
1 hr. o min.....	"	3.540	+.034	+1.016	2.986	+.023	+.135	3.015	+.026	+.108	+1.259	9.33	8.07	100
2 hr. o min.....	"	3.574	+.039	+1.166	3.009	+.036	+.212	3.041	+.042	+175.	+1.553	9.33	7.78	96
3 hr. o min.....	"	3.613	+.045	+1.345	3.045	+.035	+.206	3.083	+.042	+.175	+1.726	9.33	7.60	94
4 hr. o min.....	"	3.658	+.042	+1.256	3.080	+.037	+.218	3.125	+.027	+.112	+1.586	9.33	7.74	96
5 hr. o min.....	"	3.700	+.018	+.538	3 117	+.036	+.212	3.152	+.007	+.029	+.779	9.33	8.55	105
6 hr. o min.....	"	3.718	.000	.000	3.153	+.012	+.071	3.159	—.009	—.037	+.034	9.33	9.30	115
7 hr. o min.....	"	3.718	—.026	—.777	3.165	—.007	—.041	3.150	—.018	—.075	—.893	9.33	10.22	126
8 hr. o min.....	"	3.692	—.038	—1.136	3.158	—.027	—.159	3.132	—.035	—.145	—1.440	9.33	10.77	133
9 hr. o min.....	"	3.654	—.044	—1.315	3.131	—.037	—.218	3.097	—.040	—.166	—1.699	9.33	11.03	136
10 hr. o min.....	"	3.610	—.042	—1.256	3.094	—.034	—.200	3.057	—.034	—.141	—1.597	9.33	10.93	135
11 hr. o min.....	"	3.568	—.026	—.777	3.060	—.033	—.194	3.023	—.033	—.137	—1.108	9.33	10.44	129
12 noon.....	"	3.542	—.006	—.179	3.027	—.021	—.123	2.990	—.036	—.150	—.452	9.33	9.78	121
1 hr. o min.....	"	3.536	—.006	—.179	3.006	—.021	—.123	2.954	—.022	—.092	—.394	9.33	9.72	120
2 hr. o min.....	"	3.530	—.012	—.359	2.985	—.025	—.147	2.932	—.024	—.100	—.606	9.33	9.94	123
3 hr. o min.....	"	3.518	— 008	—.239	2.960	—.018	—.106	2.908	—.034	—.141	—.486	9.33	9.82	121
4 hr. o min.....	"	3.510	—.006	—.179	2.942	—.021	—.123	2.874	—.022	—.092	—.334	9.33	9.72	120
5 hr. o min.....	"	3.504	+.004	+.119	2.921	—.022	—.129	2.852	—.008	—.032	—.042	9.33	9.37	115
6 hr. o min.....	"	3.508	+.008	+.239	2.899	—.012	—.071	2.844	+.003	+.012	+.180	9.33	9.15	113
7 hr. o min.....	"	3.516	.000	.000	2.887	—.006	—.035	2.847	+.014	+.058	+.023	9.33	9.31	115
8 hr. o min.....	"	3.516	—.002	—.060	2.881	+.008	+.048	2.861	+.021	+.087	+.075	9.33	9.26	114
9 hr. o min.....	"	3.514	+.005	+.149	2.889	+.011	+.065	2.882	+.021	+.087	+.301	9.33	9.03	111
10 hr. o min.....	"	3.519	+.004	+.119	2.900	+.004	+.024	2.903	+.023	+.096	+.239	9.33	9.09	112
11 hr. o min.....	"	3.523	+.019	+.568	2.904	+.028	+.165	2.926	+.030	+.125	+.858	9.33	8.47	104

Hourly Variation in the Consumption of Croton Water, Shown by Rise and Fall of Reservoirs in Central Park, New York City, while Aqueduct is Delivering at a Constant Rate of Flow—Continued.

Time.		New Reservoir.			Old Reservoir.									
		Water about 3 feet deep over division-wall, so both divisions at same level.			North Division.			South Division.						
		At this average level 1.00 feet in depth contains 29.9 million gallons.			At this average level 1.00 feet in depth contains 5.88 million gallons.			At this average level 1.00 feet in depth contains 4.16 million gallons.						
Beginning of Interval.	Duration of Interval.	Mean gauge reading at beginning of interval, feet.	Rise or fall of surface during interval, feet.	Excess or deficiency of consumption over delivery from this reservoir in million gallons during this hour.	Mean gauge reading at beginning of interval, feet.	Rise or fall of surface during interval, feet.	Excess or deficiency of consumption over delivery from this reservoir in million gallons during this hour.	Mean gauge reading at beginning of interval, feet.	Rise or fall of surface during interval, feet.	Excess or deficiency of consumption over delivery from this reservoir in million gallons during this hour.	Total excess or deficiency of consumption over delivery during this hour in million gallons.	Delivery by Croton Aqueduct during this hour in million gallons.	Total consumption of water from all distribution pipes during this hour in million gallons.	Rate of consumption per inhabitant per 24 hours, including both high and low service.
SUNDAY, DECEMBER 10, 1899.														
12 midnight	1 hour.	3.542	+.023	+.688	2.932	+.040	+.235	2.956	+.042	+.175	+1.098	9.33	8.23	102
1 hr. o min.	"	3.565	+.032	+.957	2.972	+.022	+.129	2.998	+.032	+.133	+1.219	9.33	8.11	100
2 hr. o min.	"	3.597	+.034	+1.016	2.994	+.021	+.123	3.030	+.030	+.125	+1.264	9.33	8.07	100
3 hr. o min.	"	3.631	+.039	+1.166	3.015	+.033	+.194	3.060	+.057	+.237	+1.597	9.33	7.73	95
4 hr. o min.	"	3.670	+.038	+1.136	3.048	+.050	+.295	3.117	+.030	+.125	+1.556	9.33	7.77	96
5 hr. o min.	"	3.708	+.033	+.987	3.098	+.022	+.129	3.147	+.018	+.075	+1.191	9.33	8.14	100
6 hr. o min.	"	3.741	+.027	+.807	3.120	+.022	+.129	3.165	+.017	+.071	+1.007	9.33	8.32	103
7 hr. o min.	"	3.768	+.008	+.239	3.142	+.020	+.118	3.182	+.016	+.067	+.424	9.33	8.91	110
8 hr. o min.	"	3.776	—.009	—.269	3.162	+.013	+.076	3.198	+.014	+.058	—.135	9.33	9.47	117
9 hr. o min.	"	3.767	—.019	—.568	3.175	—.008	—.048	3.212	+.093	+.012	—.604	9.33	9.93	123
10 hr. o min.	"	3.748	—.021	—.628	3.167	—.011	—.065	3.215	—.022	—.092	—.785	9.33	10.12	125
11 hr. o min.	"	3.727	—.019	—.568	3.156	—.006	—.035	3.193	—.023	—.096	—.699	9.33	10 03	124
12 noon	"	3.708	—.010	—.299	3.150	—.004	—.024	3.170	—.017	—.071	—.394	9.33	9.72	120
1 hr. o min.	"	3.698	—.010	—.299	3 146	—.003	—.017	3.153	—.006	—.024	—.340	9.33	9.67	119
2 hr. o min	"	3.688	—.c06	—.179	3.143	—.001	—.006	3.147	—.004	—.016	—.201	9.33	9.53	117
3 hr. o min.	"	3 682	—.001	—.030	3.142	+.003	+.017	3.143	+.005	+.020	+.027	9.33	9.32	115

4 hr. o min.	"	3.681	+.002	+.060	3.145	+.002	+.012	3.148	+.006	+.024	+.096	9.33	9.23	114
5 hr. o min.	"	3.683	+.007	+.209	3.147	+.003	+.017	3.154	+.005	+.020	+.246	9.33	9.08	112
6 hr. o min.	"	3.690	+.005	+.149	3.150	+.005	+.030	3.159	+.010	+.041	+.220	9.33	9.11	112
7 hr. o min.	"	3.695	+.010	+.299	3.155	+.003	+.017	3.169	+.011	+.045	+.361	9.33	8.97	111
8 hr. o min.	"	3.705	+.014	+.419	3.158	+.010	+.060	3.180	+.017	+.071	+.550	9.33	8.78	109
9 hr. o min.	"	3.719	+.014	+.419	3.168	+.016	+.094	3.197	+.016	+.067	+.580	9.33	8.75	108
10 hr. o min.	"	3.733	+.022	+.658	3.184	+.016	+.094	3.213	+.017	+.071	+.823	9.33	8 51	105
11 hr. o min.	"	3.755	+.028	+.837	3.200	+.022	+.189	3.230	+.030	+.125	+1.091	9.33	8.24	101

MONDAY, DECEMBER 11, 1899.

12 midnight	1 hour.	3.783	+.036	+1.076	3.222	+.033	+.194	3.260	+.036	+.150	+1.420	9.33	7.91	98
1 hr. o min.	"	3.819	+.042	+1.276	3.255	+.034	+.200	3.296	+.037	+.154	+1.610	9.33	7.72	95
2 hr. o min.	"	3.861	+.044	+1.315	3.289	+.033	+.194	3.333	+.036	+.150	+1.659	9.33	7.67	95
3 hr. o min.	"	3.905	+.043	+1.286	3.322	+.034	+.200	3.369	+.036	+.150	+1.636	9 33	7.69	95
4 hr. o min.	"	3.948	+.037	+1.106	3.356	+.034	+.200	3.405	+.033	+.137	+1.443	9.33	7.89	97
5 hr. o min.	"	3.985	+.022	+.658	3.390	+.026	+.153	3.438	+.020	+.083	+.894	9.33	8.44	104
6 hr. o min.	"	4.007	—.003	—.090	3.416	+.009	+.054	3.458	—.007	—.028	—.064	9.33	9.39	116
7 hr. o min.	"	4.004	—.040	—1.196	3.425	—.008	—.048	3.451	—.040	—.166	—1.410	9.33	10.74	132
8 hr. o min.	"	3.964	—.050	—1.495	3.417	—.033	—.194	3.411	—.060	—.250	—1.939	9.33	11.27	139
9 hr. o min.	"	3.914	—.061	—1.824	3.384	—.036	—.212	3.351	—.056	—.233	—2.269	9.33	11.60	143
10 hr. o min.	"	3.853	—.057	—1.704	3.348	—.036	—.212	3.295	—.045	—.187	—2.103	9.33	11.43	141
11 hr. o min.	"	3.796	—.054	—1.614	3.312	—.042	—.247	3.250	—.044	—.183	—2.044	9.33	11.37	140
12 noon	"	3.742	—.041	—1.226	3.270	—.033	—.194	3.206	—.034	—.141	—1.561	9.33	10.89	134
1 hr. o min.	"	3.701	—.039	—1.166	3.237	—.034	—.200	3.172	—.034	—.141	—1.507	9.33	10.84	134
2 hr. o min.	"	3.662	—.038	—1.136	3.203	—.034	—.200	3.138	—.036	—.150	—1.486	9.33	10.82	133
3 hr. o min.	"	3.624	—.036	—1.076	3.169	—.031	—.182	3.102	—.028	—.116	—1.374	9.33	10.70	132
4 hr. o min.	"	3.588	—.038	—1.136	3.138	—.031	—.182	3.074	—.018	—.075	—1.393	9.33	10.72	132
5 hr. o min.	"	3.550	—.027	—.807	3.107	—.030	—.176	3.050	—.028	—.116	—1.099	9.33	10.43	128
6 hr. o min.	"	3.523	—.020	—.598	3.077	—.028	—.165	3.028	—.020	—.083	—.846	9.33	10.18	125
7 hr. o min.	"	3.503	—.013	—.389	3.049	—.014	—.082	3.008	—.006	—.024	—.495	9.33	9.83	121
8 hr. o min.	"	3.490	—.005	—.149	3.035	—.007	—.041	3.002	+.002	+.008	—.182	9.33	9.51	117
9 hr. o min.	"	3.485	+.018	+ 538	3.028	—.004	—.024	3.004	+.011	+.045	+.559	9.33	8.77	108
10 hr. o min.	"	3.503	.000	.000	3.024	+.002	+.012	3.015	+.012	+.048	+.060	9.33	9.27	114
11 hr. o min.	"	3.503	+.011	+.329	3.026	+.003	+.017	3.027	+.022	+.091	+.437	9.33	8.89	110

Hourly Variation in the Consumption of Croton Water, Shown by Rise and Fall of Reservoirs in Central Park, New York City, while Aqueduct is Delivering at a Constant Rate of Flow—Continued.

Time.		New Reservoir.			Old Reservoir.						Total excess or deficiency of consumption over delivery during this hour in million gallons.	Delivery by Croton Aqueduct during this hour in million gallons.	Total consumption of water from all distribution pipes during this hour in million gallons.	Rate of consumption per inhabitant per 24 hours, including both high and low service.
		Water about 3 feet deep over division-wall, so both divisions at same level.			North Division.			South Division.						
		At this average level 1.00 feet in depth contains 29.9 million gallons.			At this average level 1.00 feet in depth contains 5.88 million gallons.			At this average level 1.00 feet in depth contains 4.16 million gallons.						
Beginning of Interval.	Duration of Interval.	Mean gauge reading at beginning of interval, feet.	Rise or fall of surface during interval, feet.	Excess or deficiency of consumption over delivery from this reservoir in million gallons during this hour.	Mean gauge reading at beginning of interval, feet.	Rise or fall of surface during interval, feet.	Excess or deficiency of consumption over delivery from this reservoir in million gallons during this hour.	Mean gauge reading at beginning of interval, feet.	Rise or fall of surface during interval, feet.	Excess or deficiency of consumption over delivery from this reservoir in million gallons during this hour.				
		TUESDAY, DECEMBER 12, 1899.												
12 midnight	1 hour.	3.514	+.062	+1.854	3.029	+.014	+.082	3.049	+.029	+.121	+2.057	9.33	7.27	90
1 hr. 0 min.	"	3.576	+.040	+1.196	3.043	+.030	+.176	3.078	+.035	+.146	+1.518	9.33	7.81	96
2 hr. 0 min.	"	3.616	+.037	+1.106	3.073	+.055	+.323	3.113	+.035	+.146	+1.575	9.33	7.76	96
3 hr. 0 min.	"	3.653	+.037	+1.106	3.128	+.018	+.106	3.148	+.033	+.137	+1.349	9.33	7.98	98
4 hr. 0 min.	"	3.690	+.033	+.987	3.146	+.023	+.135	3.181	+.028	+.116	+1.238	9.33	8.09	100
5 hr. 0 min.	"	3.723	+.025	+.747	3.169	+.029	+.171	3.209	+.024	+.100	+1.018	9.33	8.31	102
6 hr. 0 min.	"	3.748	+.016	+.478	3.198	+.014	+.082	3.233	+.004	+.016	+.576	9.33	8.75	108
7 hr. 0 min.	"	3.764	—.014	—.419	3.212	—.016	—.094	3.237	—.029	—.120	—.633	9.33	9.96	123
8 hr. 0 min.	"	3.750	—.040	—1.196	3.196	—.021	—.123	3.208	—.034	—.225	—1.544	9.33	10.87	134
9 hr. 0 min.	"	3.710	—.016	—1.375	3.175	—.028	—.106	3.154	—.054	—.225	—1.706	9.33	11.04	136
10 hr. 0 min.	"	3.664	—.039	—1.166	3.157	—.018	—.106	3.100	—.033	—.137	—1.409	9.33	10.74	132
11 hr. 0 min.	"	3.625	—.048	—1.435	3.139	—.019	—.112	3.067	—.016	—.067	—1.614	9.33	10.94	135
12 noon	"	3.577	—.022	—.658	3.120	—.031	—.182	3.051	—.011	—.045	—.885	9.33	10.22	126
1 hr. 0 min.	"	3.555	—.015	—.449	3.089	—.031	—.182	3.040	—.041	—.171	—.802	9.33	10.13	125
2 hr. 0 min.	"	3.540	—.030	—.897	3.058	—.038	—.223	2.999	—.042	—.175	—1.295	9.33	10.63	131
3 hr. 0 min.	"	3.510	—.052	—1.535	3.020	—.040	—.235	2.957	—.021	—.087	—1.877	9.33	11.21	138

4 hr. o min.....	"	3.458	—.048	—1.435	2.980	—.035	—.206	2.936	—.020	—.083	—1.724	9.33	11.05	136
5 hr. o min.....	"	3.410	—.027	—.807	2.945	—.028	—.165	2.916	—.046	—.191	—1.163	9.33	10.49	129
6 hr. o min.....	"	3.383	—.021	—.628	2.917	—.025	—.147	2.870	—.019	—.079	—.854	9.33	10.18	125
7 hr. o min.....	"	3.362	—.012	—.359	2.892	—.018	—.106	2.851	—.001	—.004	—.469	9.33	9.80	121
8 hr. o min.....	"	3.350	—.003	—.090	2.874	—.012	—.072	2.850	+.007	+.028	—.134	9.33	9.46	117
9 hr. o min.....	"	3.347	+.006	+.179	2.862	—.002	—.012	2.857	+.008	+.032	+.199	9.33	9.13	112
10 hr. o min.....	"	3.353	+.013	+.389	2.860	+.002	+.012	2.865	+.007	+.028	+.429	9.33	8.90	110
11 hr. o min.....	"	3.300	+.025	+.747	2.862	+.008	+.048	2.872	+.025	+.108	+.903	9.33	8.43	104

WEDNESDAY, DECEMBER 13, 1899.

12 midnight.....	1 hour.	3.391	+.034	+1.016	2.870	+.020	+.118	2.898	+.032	+.133	+1.267	9.33	8.05	100
1 hr. o min.....	"	3.425	+.035	+1.016	2.890	+.028	+.165	2.930	+.030	+.125	+1.336	9.33	7.99	98
2 hr. o min.....	"	3.460	+.044	+1.315	2.918	+.027	+.159	2.960	+.030	+.125	+1.599	9.33	7.73	95
3 hr. o min.....	"	3.504	+.040	+1.196	2.945	+.029	+.171	2.990	+.033	+.137	+1.504	9.33	7.83	96
4 hr. o min.....	"	3.544	+.037	+1.116	2.974	+.031	+.182	3.023	+.032	+.133	+1.421	9.33	7.91	97
5 hr. o min.....	"	3.581	+.026	+.777	3.005	+.030	+.176	3.055	+.022	+.091	+1.044	9.33	8 29	102
6 hr. o min.....	"	3.607	.000	.000	3.025	+.010	+.060	3.077	—.004	—.016	+.044	9.33	9.29	114
7 hr. o min.....	"	3.607	—.032	—.957	3.045	—.021	—.065	3.073	—.040	—.167	—1.189	9.33	10.52	130
8 hr. o min.....	"	3.575	—.045	—1.345	3.034	—.025	—.147	3.033	—.053	—.221	—1.713	9.33	11.04	136
9 hr. o min.....	"	3.530	—.048	—1.435	3.009	—.027	—.159	2.980	—.053	—.221	—1.815	9.33	11.15	137
10 hr. o min.....	"	3.482	—.050	—1.495	2.982	—.031	—.182	2.927	—.039	—.162	—1.839	9.33	11.17	138
11 hr. o min.....	"	3.432	—.039	—1.165	2.951	—.037	—.218	2.888	—.038	—.158	—1.542	9.33	10.87	134
										Per cent. of addition.				
12 noon.....	"	3.393	—.043	—1.286	2.914	.....	.....	2.850	.....	34	—1 723	9.33	11.05	136
1 hr. o min.....	"	3.350	—.014	—.419	Observer at Old Reservoir quit work.					40	—.587	9.33	9.92	122
2 hr. o min.....	"	3.336	—.052	—1.555	From this time on add to the measured efflux					35	—2.100	9.33	11.40	140
3 hr. o min.....	"	3.284	—.036	—1.076	or influx to New Reservoir a percentage allowance					26	—1.355	9.33	10.68	132
4 hr. o min.....	"	3.248	—.024	—.718	for corresponding change in volume at Old					28	—.919	9.33	10.25	126
5 hr. o min.....	"	3.224	—.024	—.718	Reservoirs in same interval, determined by aver-					33	—.956	9.33	10.29	127
6 hr. o min.....	"	3.200	—.021	—.628	aging all previous observations for the same hour					40	—.879	9.33	10.21	126
7 hr. o min.....	"	3.179	—.011	—.329	.....	.....	.....	.....	.....	29	—.425	9 33	9.75	120
										36				
8 hr. o min.....	"	3.168	+.002	+.060	.....	.....	.....	.....	.....		+.082	9.33	9.25	120
9 hr. o min.....	"	3.170	+.015	+.448	.....	.....	.....	.....	.....	10	+.493	9.33	8.84	109
10 hr. o min.....	"	3.185	+.025	+.747	.....	.....	.....	.....	.....	24	+.926	9.33	8.40	103
11 hr. c min.....	"	3.210	+.033	+.987	.....	.....	.....	.....	.....	23	+1.214	9.33	8.12	100

Hourly Variation in the Consumption of Croton Water, Shown by Rise ana Fall of Reservoirs in Central Park, New York City, while Aqueduct is Delivering at a Constant Rate of Flow—Continued.

Time.		New Reservoir.			Old Reservoir.								
		Water about 3 feet deep over division-wall, so both divisions at same level.			North Division.			South Division.					
		At this average level 1.00 foot in depth contains 29.9 million gallons.											
Beginning of Interval.	Duration of Interval.	Mean gauge reading at beginning of interval, feet.	Rise or fall of surface during interval, feet.	Excess or deficiency of consumption over delivery from this reservoir in million gallons during this hour.						Per cent. of addition.	Total excess or deficiency of consumption over delivery during this hour in million gallons.	Delivery by Croton Aqueduct during this hour in million gallons.	Total consumption of water from all distribution pipes during this hour in million gallons.	Rate of consumption per inhabitant per 24 hours, including both high and low service.
THURSDAY, DECEMBER 14, 1893.														
12 midnight	1 hour.	3.243	+.041	+1.226	.....	.....	.....	.....	.....	15	+1.410	9.33	7.92	98
1 hr. 0 min	"	3.284	+.043	+1.285	.....	.....	.....	.....	.....	26	+1.618	9.33	7.71	95
2 hr. 0 min	"	3.327	+.047	+1.405	.....	.....	.....	.....	.....	32	+1.855	9.33	7.48	92
3 hr. 0 min	"	3.374	+.042	+1.256	.....	.....	.....	.....	.....	25	+1.570	9.33	7.76	96
4 hr. 0 min	"	3.416	+.036	+1.076	Note that volume for 1 foot in depth of New Reservoir is 29.9 million gallons.					27	+1.366	9.33	7.96	98
5 hr. 0 min	"	3.452	+.022	+.658						38	+.908	9.33	8.42	104
6 hr. 0 min	"	3.474	—.009	—.269	Old Reservoir {North Division 5.88 / South Division 4.16} = 10 04					22	—.329	9.33	9.66	119
7 hr. 0 min	"	3.465	—.031	—.927						37	—1.270	9.33	10.60	131
8 hr. 0 min	"	3.434	—.045	—1.345	Thus Old adds $\frac{10.04}{29.9}$ = 33.6% to New, if each					28	—1.721	9.33	11.05	136
9 hr. 0 min	"	3.389	—.043	—1.285	be drawn equal depths.					25	—1.605	9.33	10.93	135
10 hr. 0 min	"	3.346	—.040	—1.196						24	—1.484	9.33	10.81	133
11 hr. 0 min	"	3.306	—.041	—1.226	Observations December 8 to 13 show they do not rise and fall exactly alike, so we take this variable percentage as found by direct observation.					22	—1.495	9.33	10.82	133
12 noon	"	3.265	—.027	—.807						34	—1.080	9.33	10.41	128
1 hr. 0 min	"	3.238	—.034	—1.016	.....	.....	.....	.....	.....	40	—1.422	9.33	10.75	132
2 hr. 0 min	"	3.204	—.030	—.897	.....	.....	.....	.....	.....	35	—1.012	9.33	10.34	127
3 hr. 0 min	"	3.174	—.030	—.897	.....	.....	.....	.....	.....	26	—1.130	9.33	10.44	129

4 hr. o min.....	"	3.144	—.029	—.867	.....	.....	.....	.....	.....	28	—1.110	9.33	10.44	129
5 hr. o min.....	"	3.115	—.025	—.747	.....	.....	.....	.....	.....	31	—.994	9.33	10.32	127
6 hr o min.....	"	3.090	—.015	—.448	.....	.....	.....	.....	.....	40	—.628	9.33	9.96	123
7 hr. o min.....	"	3.075	—.007	—.209	.....	.....	.....	.....	.....	29	—.270	9.33	9.60	118
8 hr. o min.....	"	3.068	.000	.000	.....	.....	.....	.....	.....	36	.000	9.33	9.33	115
9 hr. o min.....	"	3.068	+.012	+.359	.....	.....	.....	.....	.....	10	+.395	9.33	8.93	110
10 hr. o min.....	"	3.080	+.020	+.598	.....	.....	.....	.....	.....	24	+.742	9.33	8.59	106
11 hr. o min.....	"	3.100	+.028	+.837	.....	.....	.....	.....	.....	23	+1.030	9.33	8.30	102

FRIDAY, DECEMBER 15, 1899.

12 midnight.....	1 hour.	3.128	+.042	+1.256	.....	.....	.....	.....	.....	15	+1.445	9.33	7.88	97
1 hr. o min.....	"	3.170	+.042	+1.256	.....	.....	.....	.....	.....	26	+1.582	9.33	7.75	96
2 hr. o min.....	"	3.212	+.044	+1.315	.....	.....	.....	.....	.....	32	+1.720	9.33	7.61	94
3 hr. o min.....	"	3.256	+.047	+1.405	.....	.....	.....	.....	.....	25	+1.755	9.33	7.58	94
4 hr. o min.....	"	3.303	+.047	+1.405	.....	.....	.....	.....	.....	27	+1.783	9.33	7.55	93
5 hr. o min.....	"	3.350	+.025	+.747	.....	.....	.....	.....	.....	38	+1.030	9.33	8.30	102
6 hr. o min.....	"	3.375	+.022	+.658	.....	.....	.....	.....	.....	22	+.802	9.33	8.53	105
7 hr. o min.....	"	3.397	—.027	—.807	.....	.....	.....	.....	.....	37	—1.105	9.33	10.43	129
8 hr. o min.....	"	3.370	—.037	—1.100	.....	.....	.....	.....	.....	28	—1.416	9.33	10.75	132
9 hr. o min.....	"	3.333	—.043	—1.285	.....	.....	.....	.....	.....	25	—1.605	9.33	10.93	135
10 hr. o min.....	"	3.290	—.055	—1.654	.....	.....	.....	.....	.....	24	—2.051	9.33	11.38	140
11 hr. o min	"	3.235	—.035	—1.046	.....	.....	.....	.....	.....	22	—1.277	9.33	10.61	131
12 noon.....	"	3.200	.....	.....	.....	.....	.....	.....	.....	..	.....	....	.....	...

RATE OF CONSUMPTION OF HIGH SERVICE WATER IN MANHATTAN.

The pump records taken at the end of each eight hour shift of the enginemen were copied and show the pumping of high service water to have been normal all of the time covered by these measurements except for the Saturday and Sunday already noted.

It was intended to also determine the curve of hourly variation in consumption for the high service, and arrangements, properly authorized by Mr. Birdsall, Chief Engineer of the Department Water Supply, were made for securing the reading of each engine counter at the beginning of every hour. These arrangements were broken up by a subordinate of the Water Department, who ordered counter readings discontinued, through a misunderstanding. The brief series of complete observations indicate such anomalous relations in day and night consumption of high service as to demand further investigation than I have found time to make. The complete investigation would require examination into slip of pumps, into the partial closing of gates in the high service distribution pipes, that has, according to my information, been executed for the purpose of lessening the pressure and so checking the consumption, and there should be made also an approximate census of consumers, number of taps and number of meters in the high service district. So far as I can learn, nothing of this kind now exists. This would have been a work beyond the scope of my instructions. Such roughly approximate figures as I have obtained regarding the high service consumption indicate a surprisingly high average consumption per capita, and an unusually high rate of draft at night; the rate of draft from 2 to 4 A. M. being not far from 90 per cent. of the mean draft for the 24 hours. These figures are so large as to suggest a more complete investigation before accepting them as giving the real draft of high service water at night, although without doubt the present severe shortage of water by day in the high service district necessitating the large use of house tanks, to be filled at night when the pressure is high, causes the night draft to continue larger than would otherwise be the case, until such hour as these tanks are refilled.

There is something unaccountable in the large high service draft. It would be profitable for the Water Department to not only follow up carefully this matter of consumption of high service water, but to make such experiments upon the rate of low service draft in different parts of the city as could be done by a manipulation of the main distributing gates, such as would throw first one district and then another on to the Old Reservoir in Central Park for its supply, and thus permit its fluctuation of draft to be noted.

CONCLUSIONS DRAWN FROM THESE CENTRAL PARK EXPERIMENTS.

Returning again to the measurement of rate of draft from hour to hour by the Borough of Manhattan as a whole, I can find in the observations themselves or in the arrangement of gates and conduits no ground for discrediting the indication of these experiments, that with an average rate of draft of 115 gallons per 24-hour day:

The rate of draft from 2 to 4 A. M., was never less than..	90	gallons per inhabitant per 24 hours.
And averaged.....	94	" " " " " "
While the highest rate of draft reached during working hours was.....	137	" " " " " "

Thus the excess of consumption in the most active hour of the day was found to be only 43 gallons above that in the most quiet hour of the night.

Late on Sunday night, or rather, very early on Monday morning, we would expect the consumption to be at its minimum, for at that time domestic use will be almost nothing, factories will be idle, house tanks and factory tanks will nearly all have become refilled through the higher water pressure of Sunday, and almost the only legitimate uses that can be thought of are the comparatively few newspaper and telegraph offices, the filling of a few steam-heating boilers in office buildings, etc., a small quantity for electric-light stations and a little for hospitals, or for an occasional household in which there is sickness; but even in these instances the use will be much smaller than on the same premises by day.

From 2 to 4 A. M. on Monday, December 4, and in the same hours on Monday, December 11, 1899, we find the measurements agree most remarkably in showing a draft at the rate of 94 gallons per inhabitant per 24 hours. The fact that this rate is the same as on other nights indicates that the tanks in commercial buildings and factories do not fail to get full before 2 A. M. on other nights than Sunday, or that there is no special industrial or commercial use through the week at from 2 to 4 A. M. different from that on Sunday.

We would hardly expect on general principles or in comparison with other cities, taking the city as a whole, that the total rate of draft for real *use* at this quietest hour of the night would be more than 10 per cent. of the increase of the most extreme day use over the smallest rate of night use—10 per cent. of 43 is 4 gallons, which, deducted from the 94, leaves 90 gallons. But the night use is without doubt larger in New York than in most other cities, and there are doubtless some tanks in the high service district especially which do not get refilled until after this hour.

There is, perhaps, a possibility that I was misinformed as to the connection being closed between the Croton high service and the Williamsbridge distribution pipes, or possibly the volume from High Service going back across the Harlem as given me by the Water Department officials may have been under-estimated, but on carefully reviewing all these chances the proof is positive that the error from these sources was not large.

To allow for all these and to further apply all margin of uncertainty from lack of precision of measurement in the direction of letting the use appear large rather than to run any risk of exaggerating the leakage, I will call the night use from 2 to 4 A. M. at the rate of 19 gallons per capita.

Deducting this *use* from the total rate of night consumption of 94 gallons, there is left, as the probable rate of waste of water at night in the Borough of Manhattan, 75 gallons per inhabitant per 24 hours. This waste is of the kind that continues at a uniform rate throughout the 24 hours, and in addition to this is the careless waste by day from leaving cocks open unnecessarily.

From this 75 gallons deduct an allowance for incurable waste of 10 gallons per inhabitant, and there is left 65 gallons per inhabitant per day of constant waste that could probably be cured.

As already stated, the result of these measurements show that one-third of the supply is used while two-thirds of the whole is wasted.

INCURABLE WASTE.

A considerable part of this waste of 75 gallons per inhabitant per day is, as a matter of practical water-works operation, absolutely incurable, escaping probably through very minute cracks in various of the half million pipe joints under ground, in so small a quantity at each joint as to necessarily escape attention.

Granting equally efficient work of the caulkers in the pipe laying gang in one city and another, we should expect that this leakage through joints and microscopic cracks would be, for a given pressure, substantially in direct proportion to the pipe diameters and the number of miles in length of the whole system of distribution pipes, and since New York contains a much larger population per mile of pipe than the average city, this joint leakage should be proportionally less.

In certain cities where all water distributed is metered in bulk, and where there is also a meter on substantially every tap, as for example, in Fall River and Woonsocket, it has been found possible to reduce this incurable leakage to 12 gallons per inhabitant per day.* Making allowance for the greater population per mile of pipe, it would appear that in Manhattan it should be possible to reduce this incurable waste to 5 gallons per inhabitant per day, but in view of the crowded and disturbed condition of many streets, the dangers of electrolysis, etc., I have, at the bottom of Diagram No. 2, drawn the line for the limit of incurable waste at 10 gallons per inhabitant per day, leaving a space on the diagram representing 65 gallons per inhabitant per day, which represents the needless waste.

To stop all of this 65 or 70 gallons of needless waste per inhabitant is

* In Milton, Mass., on 20 miles of pipes, careful study of night flow, by closing gates and supplying sections through by-pass meters, is reported to have reduced this to an almost inappreciable amount—say 1 gallon per capita or even less.

difficult and would require a meter on every tap, an elaborate, wisely administered inspection system, and great promptness in the execution of repairs, and a great deal of costly, painstaking preliminary investigation, and probably would also require the relaying of many miles of pipe and many thousand service pipes. *I believe that this great saving would be entirely practicable if public opinion would demand and support the necessary administration.*

For the purpose of illustrating that the condition of waste and use shown in Diagram No. 5 agrees in a general way with results obtained elsewhere, I have had Figure No. 8 prepared, showing similar curves of hourly variation in consumption of water obtained by Mr. Dexter Brackett with the Deacon meter in Sections 1 and 12, Charlestown District, Boston, in the year 1881, and have also shown in the lower line of Figure No. 4, the corresponding curve of use and waste for the same districts after six months' careful work with the Deacon meter and house-to-house inspection. Each of these two Charlestown Districts was a little less than equivalent to a square of 1,000 feet on a side, and contained a population averaging about 2,500 persons each, or each of these Deacon meter districts represented less than the one-thousandth part of the population whose consumption and waste of water is to be studied in Greater New York. Their records are extremely valuable, however, as showing that the method followed in estimating the waste from a curve like that of Diagram No. 3 was correct. In these efforts at waste prevention in Boston in 1881, in Section No. 12 of the Charlestown District, a night rate of 55 gallons per inhabitant found in June was reduced to 12.5 gallons per inhabitant in November by the repair of about 125 leaky fixtures in houses, the repair of leaky service pipes, the prevention possibly of some few cases of deliberate waste, and the saving of a small quantity leaking from defective mains.

The total daily consumption of 115 gallons per inhabitant per day is so well established by the Aqueduct gaugings that it is not open to doubt.

The figure of 75 gallons for waste is confirmed by noting that on subtracting this from 115 there is left for actual use 40 gallons per inhabitant per day, which is confirmed as reasonable by adding the metered water used for manufacturing and commercial purposes to the probable per capita use for domestic purposes and the probable use in public buildings.

The meter records compiled in Table No. 4 show that the total use of metered water in Manhattan and The Bronx averages 24 gallons per inhabitant per day.

Water Commissioner Dalton, page 9, Report for 1898, states that he is

informed that the 35,442 meters cover every place where water is used to any considerable extent for other than domestic purposes.

I am informed that there are a good many meters on hotels, some large tenements and apartment houses and on some of the large private residences. These measure water that should properly be charged to domestic supply—enough probably to offset the under-registry of all the meters. Moreover, some of the water that is metered and paid for is doubtless wasted.

If from the average total aqueduct delivery of		115	gallons per capita per day
we deduct for leakage, curable and incurable	75		" " " " "
and also deduct the total metered water	24		" " " " "
" " " for probable public use	3		" " " " "
		102	" " " " "
This leaves for the domestic use only		13	" " " " "
apart from all waste.			

—which may be surprisingly small, but is not unreasonably so, in view of the actual domestic consumption found in those cities in which substantially all the taps are metered and an account kept of the metered domestic consumption apart from the rest.

The following are examples:

In Fall River in 1891, the average consumption of 70,000 inhabitants, as shown by meter readings, was	11.2	gals. per capita per day.
In Fall River in 1899 the average consumption of 100,000 inhabitants, as kindly compiled for me in the Water Registrar's office by adding up the domestic meter readings, was	14.6	"
In Woonsocket, R. I., the average domestic consumption of about 27,500 inhabitants, as very kindly compiled for me from the meter readings by Mr. Byron Cook, Superintendent, was,		
In 1899	14.1	"
In 1898	13.4	"
In Worcester, Mass., for the year 1899, the total drawn through domestic meters by a population of 112,000 averaged	14.0	"
In Worcester in 1892 it averaged	16.8	

These figures on domestic consumption per capita from Fall River, Woonsocket and Worcester included, in addition to the real use, such leakage as a man permits to continue when he has a water meter in the cellar to quicken his memory and his interest in the repair of leaks.

A careful analysis of meter readings on many hundred houses, made by Mr. Dexter Brackett while engaged in investigations preliminary to the work of the Massachusetts Metropolitan Water Board, showed that the per capita consumption of water was very much smaller in the houses of the poor, or the houses with a single faucet, than in the houses of the well-to-do or wealthy. For example, the average of 619 families in Newton, Mass., in the cheapest grade of houses having but a single faucet each, showed an average consumption of only	7.	gals. per capita per day.

The average of 148 families in Fall River, in an average grade of house having commonly a bath-tub and a water-closet, averaged	8.4 gals. per capita per day.
The average of 339 apartment-houses in Boston supplied through meters showed	35.6 "
While 31 high-cost apartment-houses showed an average consumption through meters of	59. "

In studying average values or the readings of sample meters, where every tap throughout the city is not metered, there is danger that due weight will not be given to the preponderance in number of the class of inhabitants who, apart from waste, use small quantities of water.

From all of the foregoing illustrations it appears that the average of 13 gallons per capita indicated by the computation on page 28 is not entirely unreasonable.

The soundness of the above-described method for determining this hourly variation in the consumption of water is grounded in the following propositions:

1st. The New Croton Aqueduct was supplying all of the water that fed the distribution pipes on Manhattan Island and supplied it with almost absolute uniformity of flow.

2d. The only equalizing basins large enough to affect the measurement appreciably between the outlet of the aqueduct and the consumers' taps are the reservoirs in Central Park.

The Central Park reservoirs were in very free communication with the aqueduct.

3d. Substantially all of the water which the aqueduct delivered in excess of the rate of consumption went into Central Park Reservoirs (it had no other place to go).

All of the excess of draft at any hour over supply came from the Central Park Reservoirs (there was no other place it could come from).

Thus it made little difference whether or not some of the water was diverted into the distribution between the Harlem River and Central Park without passing into or out of these reservoirs.

The uncertainties are:

1st. The fluctuations in the small high-service reservoir (of 1 acre in area against the 125 acres of water in Central Park).

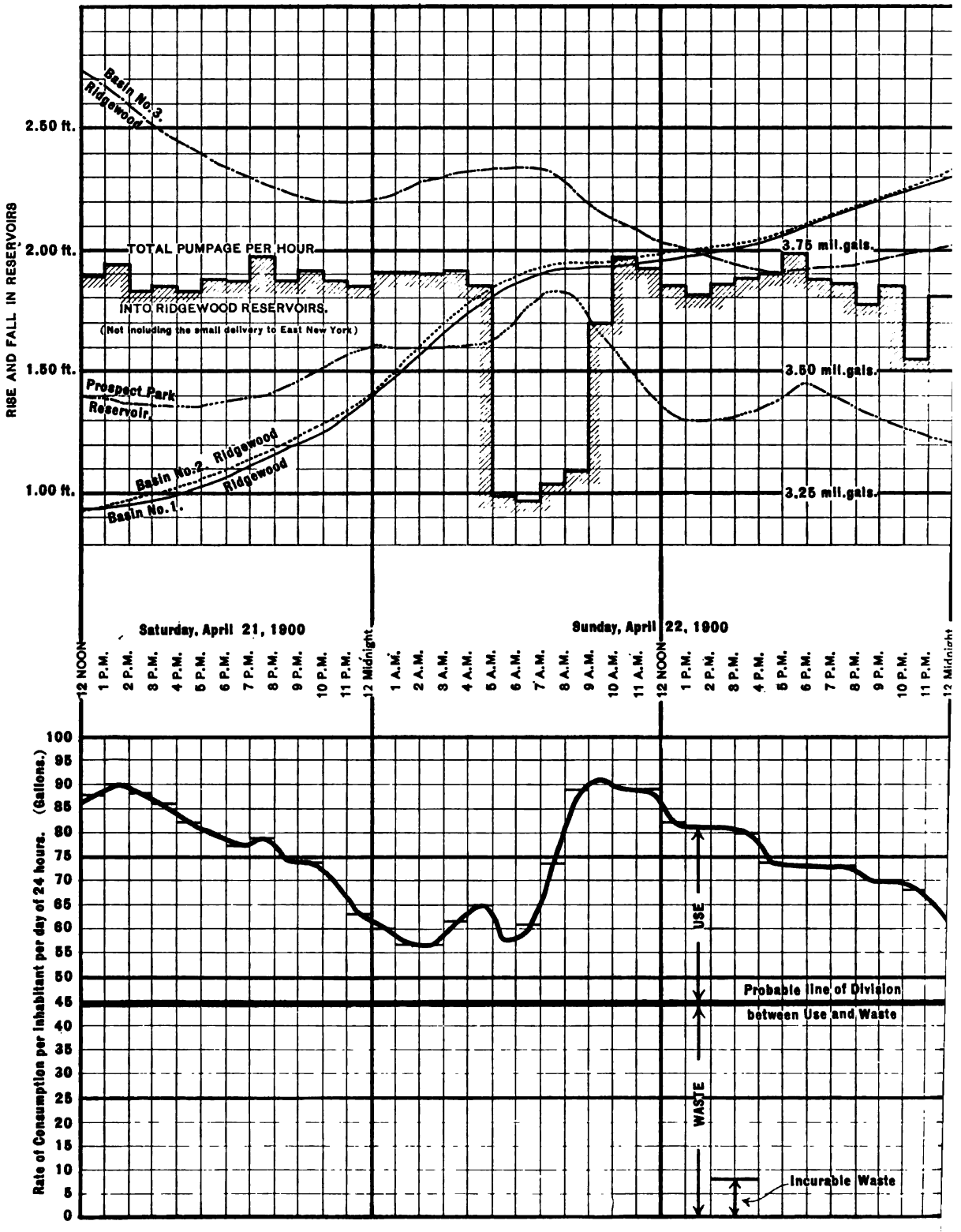
2d. The possibility that the unaccountable uniformity of high service pump delivery may be in part due to an overflow at night from high service into the Bronx distribution pipes with Williamsbridge Reservoir as an equalizer for both. From questioning Mr. Birdsall as to the arrangement of gates and the pipes in existence, it appears certain that this quantity, going to the Bronx, must have been so small as to not affect the results appreciably.

3d. The subdivision of the number of the population supplied with Croton water from those supplied with water from the Bronx conduit.

4th. The real use of water from 3 to 4 A. M.

5th. The quantity taken to refill house tanks after 2 A. M.

After reviewing this margin of uncertainty I am satisfied that it is covered and more than covered by increasing the allowance for rate of use at 3 A. M. from a rate of 4 to a rate of 19 gallons per capita per 24 hours.


RISE AND FALL IN RESERVOIRS
2.50 ft.
2.00 ft.
1.50 ft.
1.00 ft.
Basin No.3. Ridgewood
TOTAL PUMPAGE PER HOUR
INTO RIDGEWOOD RESERVOIRS.
(Not including the small delivery to East New York)
Prospect Park Reservoir.
Basin No.2. Ridgewood
Basin No.1. Ridgewood
3.75 mil.gals.
3.50 mil.gals.
3.25 mil.gals.
Saturday, April 21, 1900
Sunday, April 22, 1900
12 NOON
1 P.M.
2 P.M.
3 P.M.
4 P.M.
5 P.M.
6 P.M.
7 P.M.
8 P.M.
9 P.M.
10 P.M.
11 P.M.
12 Midnight
1 A.M.
2 A.M.
3 A.M.
4 A.M.
5 A.M.
6 A.M.
7 A.M.
8 A.M.
9 A.M.
10 A.M.
11 A.M.
12 NOON
1 P.M.
2 P.M.
3 P.M.
4 P.M.
5 P.M.
6 P.M.
7 P.M.
8 P.M.
9 P.M.
10 P.M.
11 P.M.
12 Midnight
Rate of Consumption per Inhabitant per day of 24 hours. (Gallons.)
100
95
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
USE
Probable line of Division between Use and Waste
WASTE
Incurable Waste

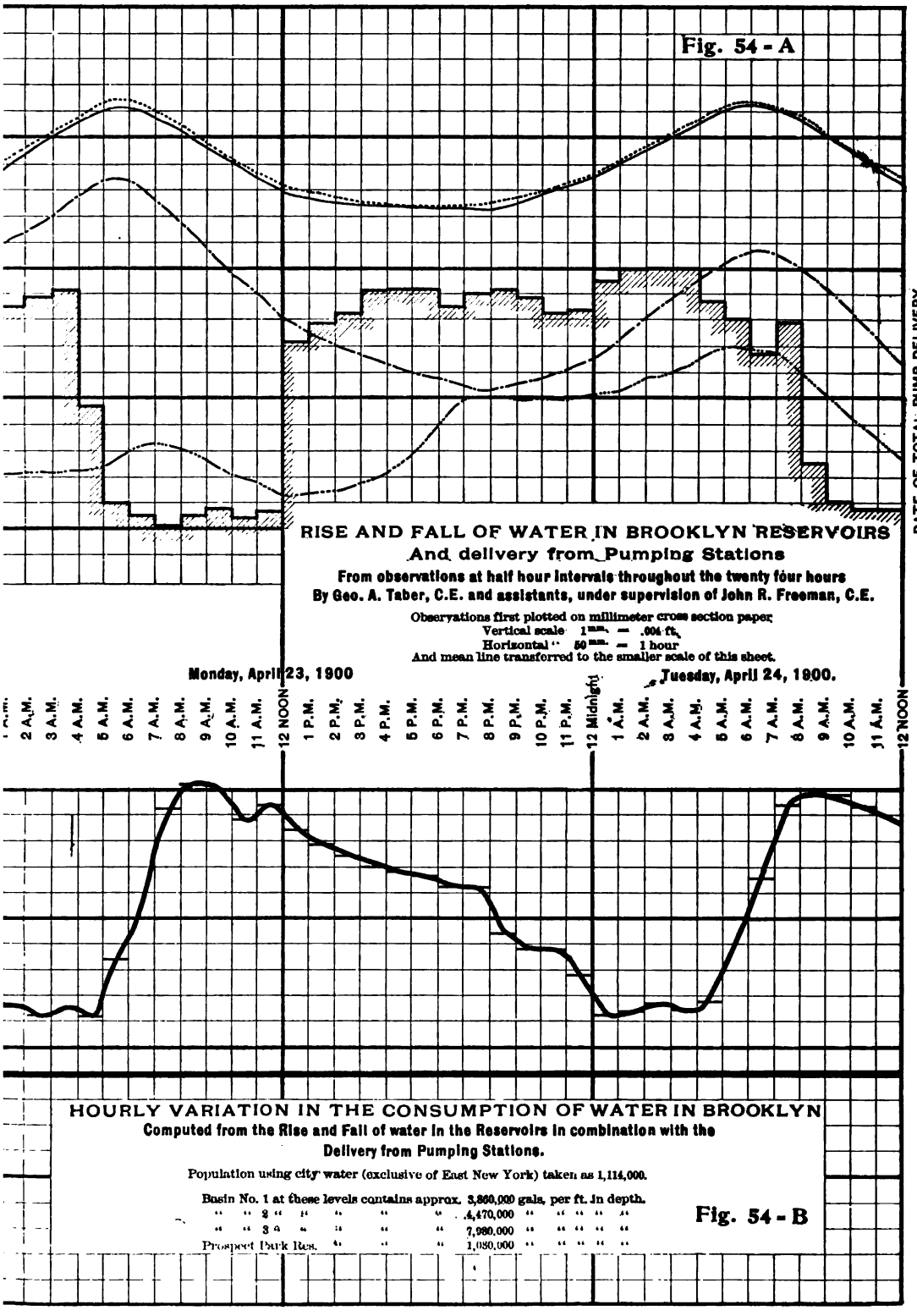
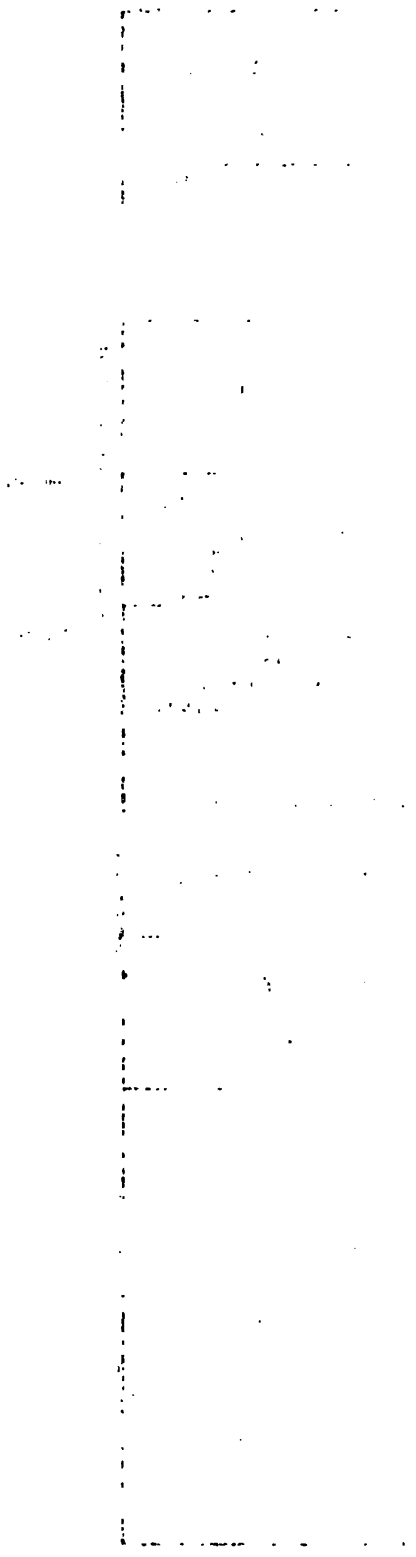


Fig. 54 - A
RATE OF TOTAL PUMP DELIVERY
5.59 MIL. GAL.
RISE AND FALL OF WATER IN BROOKLYN RESERVOIRS
And delivery from Pumping Stations
From observations at half hour intervals throughout the twenty four hours
By Geo. A. Taber, C.E. and assistants, under supervision of John R. Freeman, C.E.
Observations first plotted on millimeter cross section paper
Vertical scale 1 mm. = .004 ft.
Horizontal " 50 mm. = 1 hour
And mean line transferred to the smaller scale of this sheet.
Monday, April 23, 1900
Tuesday, April 24, 1900.
12 NOON
12 Midnight
12 NOON
HOURLY VARIATION IN THE CONSUMPTION OF WATER IN BROOKLYN
Computed from the Rise and Fall of water in the Reservoirs in combination with the
Delivery from Pumping Stations.
Population using city water (exclusive of East New York) taken as 1,114,000.
Basin No. 1 at these levels contains approx. 3,860,000 gals. per ft. in depth.
" " 2 " " " " " 4,470,000 " " " " "
" " 3 " " " " " 7,980,000 " " " " "
Prospect Park Res. " " " 1,080,000 " " " " "
Fig. 54 - B



HOURLY VARIATION IN THE RATE OF CONSUMPTION OF WATER IN BROOKLYN—PROBABLE PROPORTION OF THE SUPPLY WASTED.

A comparison of the rate of draft after midnight, when the legitimate use is small, with the rate of draft of water during the busy hours of the day gives here, as in Manhattan, the best measure that can now be obtained of the proportion actually used and of the portion of the supply wasted.

The method of measurement followed in this Brooklyn test was substantially the same as that followed in the Central Park experiments on the consumption of Croton water described in pages immediately preceding, with such variation as required by the different local conditions.

The entire Borough of Brooklyn excepting Wards 29, 30, 31 and 32, and a part of Ward 26, is supplied from the system of 3 conduits that lead in from the eastward and deliver their water at a very low elevation to the two large pumping stations at Ridgewood, about half a mile southerly from the elevated Ridgewood Reservoirs. These consist of 3 large basins, Nos. 1, 2 and 3, of areas of 11.82 acres, 13.75 acres and 24.49 acres respectively. Out from Basins 1 and 2 run pipes, united just outside, which supply the high-service pumping station for Prospect Hill; 2 of these Prospect Hill pumps deliver into the high-service reservoir at Prospect Park, of 3.32 acres water area; 3 of these pumps at the Prospect Station are for extra high service and deliver into a standpipe of about 16 feet diameter.

Therefore, if we measure accurately the quantity of water delivered in each hour of the 24 by the main pumps at the Ridgewood Station, all water which is drawn out in any given hour for consumption, use or waste, in the entire city (excluding those wards mentioned above), must unquestionably come either from the delivery of the main pumps at Ridgewood during that hour or from the lowering of the water surface in one or another of the four reservoirs:—and in any hour when the delivery of these Ridgewood pumps exceeds the consumption, the entire excess must go into one or another of these four reservoirs. (The extra-high-service standpipe is of such small area that its fluctuations may be disregarded.)

By means of measuring the hourly delivery of each of the Ridgewood pumps and by simultaneously measuring the depletion or storage of excess of water in these four reservoirs we have a positive measurement of the actual draft of water from the distribution pipes during that hour.

For the purpose of eliminating the effect of any chance errors of observation the revolution counters upon all the pumps were read each half hour

and the water level in each of the 3 Ridgewood basins was very carefully measured once each half hour. At the Prospect Park Reservoir the observer had more leisure, and so these observations were made once in each six minutes.

Two sets of observers were employed on the measurements of reservoir height, working in 12-hour shifts, and the observations continued day and night, from Saturday noon, April 21, to Tuesday noon, April 24, and would have been continued longer had time permitted. The observations on engine counters were made by the engineers in charge of the pumps.

All these observations of pump counters were plotted in the form of half-hour differences on a large scale for the purpose of detecting any chance errors in time or count, but were in general found consistent and satisfactory. The reservoir observations and also the pump observation were all first plotted on a very large scale and the mean line transferred to the accompanying drawing. To avoid confusion of lines on the diagram only the line of aggregate delivery of the eight pumps in use at Ridgewood is plotted. For measuring the fluctuations of reservoir level at Basin No. 1, a glass tube gauge with scale divided in hundredths of a foot, connected to a blow-off pipe, served admirably for avoiding effect of any wind waves. At Basin No. 2 the heights of water were measured in a still-box about 1 foot square by 2 feet 6 inches deep, communicating with the reservoir through a pipe not far from $\frac{3}{4}$ inch diameter. Check observations were also taken by means of the float gauge connecting with a steel tape divided to 1/100 foot.

At Basin No. 3 a part of the observations were made with a portable-hook gauge in a gate-chamber having free communication with reservoir, the location being between the reservoir and the screens. A part were taken by means of a scale board graduated to 1/100 foot, set in the same location.

At the Prospect Park Reservoir a glass-tube gauge connected to the blow-off pipe was used; this gauge, like the tube gauge at Basin No. 1, being a part of the regular outfit of the Brooklyn Water Department. There was very little wind during these observations and the conditions were very favorable for precise measurement. There was a slight shower on Saturday night, but weather was mild and otherwise favorable.

There are 4 Venturi meters in the 4 force mains from the North Ridgewood Pumping Station. It was attempted to use these as a check on the pump delivery measurement, but the meters were found over-registering and temporarily unreliable.

There are also 3 Venturi meters on the 3 mains leading out of the high-service reservoir and the high-service standpipe. These were also found out of order, which explains our not using them for check observations.

Data for Per Capita Basis Brooklyn Consumption.

Observations on the rate of consumption of water at different hours of the day and night appear more instructive when reduced to the per capita basis, especially when comparing the consumption of one city with another.

To obtain the population of that part of Brooklyn supplied from the Ridgewood pumps, we had recourse to the following data:

First, the total population of the Borough of Brooklyn was taken as estimated by the Department of Health for January 1, 1900, viz.: 1,249,226.

But not all of this population is supplied from the Ridgewod works.

In order to obtain the proper deduction for the population in those wards not supplied from the Ridgewood works, we find the "Eagle Almanac" gives the population (as compiled November 1, 1899, by Edwin Bolitho, of the Department of Taxes and Assessments), in the wards not supplied—Nos. 29, 30, 31 and 32—as amounting to about 6.3 per cent. of the entire population. Making this deduction, we have left about 1,114,000 as the population supplied from Ridgewood.

To compensate for the small draft of water from Ridgewood into a part of the Twenty-sixth Ward, a constant deduction of 39,000 gallons per hour was made from the Ridgewood pumpage, this being the average amount so drawn according to official records. The total of this water diverted to Ward 26 from Ridgewood is only about one per cent. of the total Ridgewood pumpage, and therefore any fluctuation in its rate of draft would be inappreciable upon the variation in consumption by the city as a whole.

It has not appeared worth the space to reproduce the tables of measurements and computed results, since these results are given with tolerable completeness on the diagram.

The methods of observation and of computation have been fully described in the case of the Central Park observations. After the computations were finished it was noted that the total consumption of water per capita was smaller than that published in the Report of the Department of Water Supply for the year 1898. The figures were therefore carefully reviewed but no error found. A subsequent inquiry at the office of the Chief Engineer showed that the actual total consumption had really fallen off due to some restriction of waste following the fear of a water famine last summer.

Brooklyn High Service Water Consumption.

It has not been found possible to obtain an estimate of the population supplied from high service and extra high service suitable for plotting the hourly variation of consumption of high-service water upon a per capita basis.

Observations were made upon its total amount in gallons per hour coincident with these other observations and are all in satisfactory form for Saturday and Sunday, but after Monday morning, April 23, a disturbance was caused by the shutting down of one of the high-service pumps for repairs and by the change of 62 gates on the distribution pipes on the boundary between high service and low service, thus temporarily throwing a part of the high-service district on to low-service.

ADDITIONAL DATA FOR ESTIMATING THE INCURABLE LEAKAGE.

The tightness reported for Milton, Mass., on page 60 was so very remarkable that I requested permission for an assistant to make some observations. This was very courteously granted by the President of the Milton Water Company, Mr. Ellerton P. Whitney, and the Superintendent of the works co-operated earnestly.

A new 4-inch crown meter, fresh from the factory and which it was said had just previously been adjusted and tested, was set in the by-pass on the pipe through which the entire town is metered and the gates so fixed that during the night the entire flow passed through this meter. This was read at 5-minute intervals for 24 hours. This meter was insufficient to pass the day flow, which was in part measured by a 6-inch union meter, which is known to be somewhat worn and out of adjustment. We are informed there have now been no leakage tests by cutting pipe system into small sections for about a year.

The night flow from 2 A. M. to 5 A. M. held very steadily at about 3,100 gallons per hour, and in no 15-minute period was the delivery notably smaller than 50 gallons per minute.

This corresponds to $50 \times 60 \times 24 =$	72,000.	gallons per 24 hours
Which is $\frac{72000}{32} =$	2,250.	gallons per 24 hours per mile of pipe
Which is equivalent to	281.	gallons per 24 hours per mile per inch in diameter
Which again is equivalent to $\frac{72000}{37000} =$	1.95	gallons per 24 hours per foot of leaded joint.

The above draft included whatever use and waste was going on from 3 to 4 A. M. along 32 miles of pipes and from 818 service pipes, and probably covered some use for washing carriages by a "garden hose" in one large stable; some use at a large milk farm in washing cans; and some use in a large chocolate factory; the exact amount of these uses not being known. The population per mile of pipe is very sparse; total population being 6,500, or only 210 per mile, and thus less than one-tenth the density per mile of pipe of New York or Brooklyn.

The rate of night use and waste per capita was 11 gallons, but the per capita basis is evidently not fair for comparison because of the sparseness of population per mile of pipe.

The total piping was—2 miles, 4-inch; 18 miles, 6-inch; 5 miles, 8-inch; 3 miles, 10-inch; 4 miles, 12-inch; total, 32 miles, and the total length of leaded pipe-joint is not far from 37,000 feet or 7 miles, or just 10 times the length of leaded pipe-joint per capita that exists in New York (see page 61).

If we assume that $\frac{2}{3}$ of this night flow was waste from cast-iron mains and $\frac{1}{3}$ from house fixtures and use it gives a pipe-waste of 1,500 gallons per 24 hours per mile of pipe, equivalent to 1.3 gallons per 24 hours per foot of leaded joint. Works 10 years old. Average pressure, 85 pounds per square inch.

Since the New York pipes contain 0.6 feet of leaded joint per capita, the total New York pipe leakage per capita on this Milton basis would be less than 1 gallon per capita per 24 hours.

I have a report of some carefully made tests on certain large new pipes which show about 4 times the average rate of leakage just quoted. There is a question if some of these leaks in new pipe will not silt up or rust up.

Mr. C. F. Loweth, Civil Engineer, of St. Paul, Minn., has kindly given me data from tests of 9 small waterworks where leakage was measured by lowering of water surface in an elevated tank. Pipes mostly 6-inch and 8-inch—pressure about 50 pounds—prior to the tapping in of service-pipes. From these tests and others Mr. Loweth finds that, under good contract work, it is possible to obtain joints so tight under 50 pounds pressure that the leakage per 24 hours per mile of pipe, per inch in diameter, will, when new, not exceed 60 to 70 gallons. One mile of 8-inch pipe, with its gates, crosses and hydrant branches, will commonly contain about 500 joints each, 2.36 feet long, or a total of 1,180 feet of leaded joint—a leakage of 70 gallons per inch mile is therefore equivalent to $\frac{70}{148}$ feet=0.47 gallons per 24 hours per foot of leaded joint, or, according to Mr. Loweth's experience, the leakage of a mile of new 8-inch cast-iron pipe need not exceed 480 to 560 gallons per 24 hours.

At this rate the incurable pipe-leakage for New York would be only about ½ gallon per capita; but I regard this, or anything near it, impossible of attainment in the conditions found in New York, and am inclined to look at 5 gallons as the ideal, and more likely 10, with the new danger of electrolysis.

Appendix No. 4.

PROBABLE INCREASE IN POPULATION.

The margin of uncertainty in the number of future inhabitants to be supplied with water is much smaller than the uncertainty in the number of gallons of water per inhabitant per day that must be provided for them to use and waste.

The future population for ten, twenty or, perhaps, thirty years to come, can be estimated with a degree of certainty sufficient for all questions of future water supply by a study of the growth for the past thirty years, and by the further consideration that the great commercial centre of our country, reflecting as it does the progress of the nation as a whole, must move onward with a more definite, uniform and persistent force than the growth of a smaller city dependent upon the prosperity of a few lines of industry or a limited commerce.

As will be seen from statistical tables which follow, the population of the district within fifteen miles of the New York City Hall in the decade from 1870 to 1880 increased 32 per cent.; in the decade from 1880 to 1890 43 per cent., and if the official estimates of growth for the past five years prove correct when the census of June, 1900, is completed, the growth for this ten years, from 1890 to 1900, will have been 33 per cent. The average increase per decade for the twenty-five years between the census of 1870 and the census of 1895, the two best points for defining the curve of past growth, is 36 per cent.

Steady and rapid progress appears more firmly grounded to-day than ever before, under the commercial developments of the past few years and with the recent marvellous growth in the export of manufactured products; but on the other hand, it seems too much to expect that growth by geometric progression in any such large ratio as 36 per cent. per decade can continue for many years.

Our most instructive example for study of growth of a great commercial centre according to the geometric ratio, is greater London; which has now for fifty years steadily maintained a rate of growth of about 20 per cent. per decade.

PER CENT. OF INCREASE IN POPULATION PER DECADE.

After plotting upon a diagram the rates of growth of our leading American cities, and those for London, Paris and Berlin,* while these all indicate in a general way that 20 per cent. increase per decade is the smallest reasonable expectation for the growth of New York in the near future, the only lines which appear really trustworthy as a guide for projecting the line of growth for New York City and its surroundings, are the lines for London and for New York itself. In other great cities, political, local or sectional influences have been at work—in Paris, the retardation due to the Franco-Prussian War; in Berlin, the fostering care of a strong centralizing government; in Chicago, the development of a point of distribution and collection of a great internal commerce, in the formative period of modern transportation—while in London we have the development of a great commercial centre under conditions which thirty-two years ago, in 1867, when London had the same population which New York and its immediate neighbors now possess, did not, as we look back upon that period and note the forces at work, bear such promise of growth or give such facilities for feeding and housing and transporting a multitude as is to be found for the near future in New York.

With greater concentration of office work in tall buildings, with the commercial, manufacturing and residential districts expanding upward as well as outward, with the greater centralization of capital and commerce, with exports of manufactures growing by leaps and bounds, and with the new importance of this country at large before the world, it surely looks probable that the growth of population in the Metropolitan District in the next ten and twenty years will be as much as in the past ten and twenty years. The line drawn on Diagram No. 13 from the statistics of London, proves that a city does not necessarily have its rate of growth diminished below 20 per cent. per decade merely because it has become large.

After considering these various matters and in view of the fact that an increase of 36 per cent. per decade has now actually been maintained for twenty-five years, with no sign of any adverse influence yet above the horizon, it appears to me not unreasonable to provide water supply for 30 per cent. increase of population in the next decade, 25 per cent. in the next, and 20 per cent. in the next decade, from 1920 to 1930.

* Figures taken from F. P. Stearns' study of growth of Boston Metropolitan District.

In Diagram No. 13, see page 87, I have plotted the line of actual growth of each group of boroughs and for the metropolitan district as a whole for the past, and have projected the lines into the future on four different assumptions.

1st. That in the next ten, twenty and thirty years, just as many inhabitants will be added as in the past ten, twenty and thirty years. This is an increase in simple arithmetical progression, and is shown by the dotted line.

2d. An increase in geometric ratio at 20 per cent. growth per decade, the same as London.

3d. That the per cent. of increase will be the same for each future decade that it averaged for the twenty-five years from the U. S. Census of 1870 to the State Census of 1895, or 36 per cent. per decade; this being an increase in geometrical progression, which is the ratio by which increase of population is commonly estimated.

4th. The line of increase which I regard as most probable, viz.:

That the growth from 1900 to 1910 will be 30 per cent.
" " " 1910 to 1920 will be 25 per cent.
" " " 1920 to 1930 will be 20 per cent.

OFFICIAL ESTIMATE OF PRESENT POPULATION.

These lines are started (with the allowance described below) from the official estimate for the city prepared in the Department of Health for January 1, 1900, which, as kindly given me by Roger S. Tracy, M. D., Registrar of Records, is as follows:

Borough of Manhattan	1,980,223
The Bronx	168,867
Brooklyn	1,249,226
Queens	133,366
Richmond	68,254
Total for Greater New York	3,599,936

This official estimate is understood to be made up from the census of 1895 for Manhattan and Bronx, and from the census of 1892 for Brooklyn by assuming the increase to have proceeded in geometrical progression from the U. S. Census of 1880, and is presented by the Department of Health subject to revision after the Federal Census is taken, a few months hence.

There is some reason to question if the growth in these recent years of commercial depression (1895 to 1898) has been so large as is assumed in this method of computation followed by the Department of Health, and I made some efforts, without success, to obtain a check computation by the annual growth in registry of school children, the annual growth in registry of voters,

and the yearly number of building permits, and by the change in number of names in the city directory from year to year; methods of estimating growth that have been found of use in other cities. Certainly the increase in use and waste of water in those years gives no suggestion that we would err in using these Health Department estimates of growth in population as part of our basis for estimating the requirement for water; but to be conservative, I have, as will be noted in the diagram, assumed that the recent "hard times" have cost the municipal district the equivalent of more than a year of normal growth.

The data kindly furnished by various officers of the Health Departments of New York and of Hudson County, New Jersey, together with that compiled from other New Jersey county reports and from the United States census reports, is presented in the following tables, Nos. 38 to 44.

The growth of the adjacent New Jersey towns is presented in these statistical tables, because in considering the influences which promote growth we must recognize that these apply to a whole district, and are not confined by strictly municipal lines. For the present purpose the Metropolitan District has been assumed to include all municipalities of which the main portion lies within fifteen miles of the New York City Hall.

ESTIMATES ADOPTED FOR FUTURE POPULATION.

The main questions are:

1st. What will be the future population of the Metropolitan District as a whole, including these New Jersey towns?

2d. In what portion of the district will the increase congregate?

This can be foreshadowed with accuracy sufficient for purposes of water supply, and is essential to the proper location of the future main arteries of the system and the future main points of distribution.

With rapid transit toward the Bronx now assured, and its large, sparsely populated areas thus made more available; with additional bridges and tunnels to Brooklyn, and with quick and easy transit assured to the vacant territory of Queens, it appears probable that Manhattan, Bronx, Brooklyn and Queens will each secure at least its full pro rata of the additional population for the entire Metropolitan District.

After Diagram No. 13 was drawn, and its lines projected on the basis of each borough sharing pro rata in the increase of population, I was led by the recent discussion of new bridges and tunnels across the East river to give

Fig. 55

OUTLINE MAP
Showing Ward Limits in 1895
in
NEW YORK & BROOKLYN
also
Population Districts
as shown on Map No. 12.

To accompany report of
J.R.Freeman, C.E.

further attention to the influence of these improved facilities for rapid transit across the East river upon the growth of Brooklyn and Queens, to project the line for Brooklyn and Queens at a higher rate, and to assume that the larger growth in this borough would be at the expense of growth in the New Jersey towns rather than by diversion from the Bronx. In Richmond the line is projected according to some consideration of its ample water front, its nearness to the centre of business, and its large, sparsely settled areas and its many excellent locations for large manufacturing plants, with attractive home-sites near at hand, and of the probable future conditions, more than by the ratio of growth in the past.

Reference is made to Diagram No. 13 for the figures adopted for the future.

While I am disposed to regard the evidence as pointing to the 30-25-20 line of future growth with stronger probability than to any of the other lines, I have, in the desire to be conservative, labeled this line as "Largest Probable," and have labeled the 20-per-cent.-per-decade line as the "Smallest Probable."

In Table No. 5, where these estimates of population are made use of for computing the magnitude of the water supply that ought to be provided for the future, a mean, in round numbers, between the 30-25-20 line, and the 20-per-cent.-per-decade line has been adopted for Manhattan and The Bronx. For Brooklyn and Queens, the 30-25-20 line is used.

Statistics of population in past years, so complete as desired for the present purposes, were difficult to obtain, and the record is somewhat confused by the various changes in municipal boundaries and subdivisions. We were in many cases unable to follow these figures for population back to their original source for verification. That the data for Table No. 44 may be more clearly set forth and made easier of verification and amendment, the following Tables 38 to 43 are presented.

Although some of the different enumerations do not agree very well, notably the Federal census of 1890 and the police census of 1890, the final result, in the aggregate, must be correct within a small margin.

See Diagram No. 13 and Table No. 5 for conclusions.

TABLE No. 38.

Actual Increase in Population of Manhattan and The Bronx by Wards.

Data mainly from Report of Board of Health for 1898. (Unpublished copy.)

Group on Fig. 49.	Ward.	United States Census, June 1, 1870.	Per Cent. of Growth in 5 years.	New York State Census, June 1, 1875.	Per Cent. of Growth in 5 years.	United States Census, June 1, 1880.	United States Census, June 1, 1890.	Per Cent. Growth by U. S., 1880 to Pol. 1890.	Police Census, 1890, Taken Sept. Corrected to June 1.	New York State Census, Feb. 1, 1892.	Per Cent. of Growth in 5 years, 1890-1895.	Police Census, April 1, 1895.
A	First	14,463	....	14,298	....	17,939	11,122	....	12,075	.....	....	12,508
	Second	1,312	....	1,012	....	1,608	929	....	1,510	.....	....	1,038
	Third	3,715	....	2,874	....	3,582	3,765	....	3,418	.....	....	4,014
	Fourth	23,748	....	20,828	....	20,996	17,809	....	19,337	.....	....	18,405
	Fifth	17,150	....	15,951	....	15,845	12,385	....	12,949	.....	....	10,603
	Sixth	21,153	....	19,861	...	20,196	23,119	....	23,058	.....	....	22,897
	Total, Group A	81,541	—8.22	74,824	7.13	80,166	69,129	—9.75	72,347	.....	—4.00	69,465
B	Seventh	44,818	....	45,636	....	50,066	57,366	....	62,139	.....	...	74,227
	Tenth	41,431	....	41,757	....	47,554	57,596	....	64,076	.....	....	70,168
	Eleventh	64,230	....	63,855	....	68,778	75,426	....	83,337	.....	....	86,722
	Thirteenth	33,364	....	34,013	....	37,797	45,884	....	51,649	.....	....	58,802
	Fourteenth	26,436	....	26,453	....	30,171	28,094	....	30,752	.....	....	31,904
	Seventeenth	95,365	....	101,075	....	104,837	103,158	....	107,737	.....	....	114,727
	Total, Group B	305,644	2.34	312,789	8.45	339,203	367,524	17.80	399,690	.....	9.23	436,550
C	Eighth	34,913	....	32,465	....	35,879	31,220	....	41,890	.....	....	31,374
	Ninth	47,609	....	49,403	....	54,596	54,425	....	60,213	.....	....	60,987
	Fifteenth	27,587	....	25,529	....	31,882	25,399	....	32,707	....	....	26,216
	Total, Group C	110,109	—2.46	107,397	13.90	122,357	111,044	10.20	134,840	.....	—12.05	118,577

D	Sixteenth	48,359	....	48,235	....	52,188	49,134	....	61 419	.....	....	57,430
	Eighteenth	59,593	....	61,195	....	66,621	63,270	....	70,299	.....	....	67,469
	Twentieth	75,407	....	79,764	....	86,015	84,327	....	93,844	.....	....	94,969
	Twenty-first	56,703	....	58,831	....	66,536	63,019	....	78,689	.....	....	72,144
	Total, Group D	240,062	3.33	248,025	9.82	272,350	259,750	11.70	304,251	.....	4.03	292,012
E	Nineteenth	86,090	....	118,727	....	158,191	234,846	....	357,766	.....	....	267,076
	Twenty-second	71,349	....	83,420	....	111,606	153,877	....	184,979	.....	....	194,893
	Total, Group E	157,439	28.40	202,147	33.40	269,797	388,723	64.00	442,745	.....	4.35	461,969
F	Twelfth	47,497	27.40	60,510	35.20	81,800	245,046	237.00	275,587	.....	32.30	364.412
Total for Borough of Manhattan		942.292	6.73	1,005,692	15.80	1,164,673	1,441,216	39.90	1,629,460	.....	6.97	1,742.985
	Twenty-third	28,981	....	24,320	....	28,338	53,948	....	60,445	.....	....	81,567
	Twenty-fourth		....	11.874	....	13,288	20,137	....	20,812	.....	....	26,508
	Annexed District, east of Bronx river	(?)	....	(?)	....	(?) 6,000	(?) 10,000	....	(?) 10,000	.....	....	17,000
Total, Borough of The Bronx		28,981	24.90	36,194	31.60	47,626	84,085	91.50	91,257	.....	37.00	125,075
Total, Manhattan and The Bronx.		971,273	7.27	1,041,836	16.30	1,212,299	1,525,301	42.00	1,720,717	1,801,739	8.57	1,868,060

In the table above and in Fig. 49 the wards having similar characteristics have, so far as practicable, been grouped together for the purpose of studying the fact that, while population in that portion of New York City below Fourteenth street has remained about stationary for 30 years, and indeed decreased below Canal street, the consumption of water in the same district has largely increased. Fig. 49 shows these group and ward lines. A continuation of this study, with a compilation showing the total of metered water in each district and a gauging of the total supply delivered into each district, would be of value in the search for waste.

The ward lines in Manhattan have not been changed within period covered above.

Previous to 1874 New York City comprised only Manhattan Island.

On January 1, 1874, all that part of present Borough of The Bronx lying west of Bronx river was annexed to New York City.

On June , 1895, all that part of present Borough of The Bronx lying east of Bronx river was annexed to New York City, the population being estimated at 17,000. (No pipes of New York's public water supply yet reach this territory east of Bronx river.) The "Greater New York" Act of January 1, 1898, thus added no new territory to Manhattan or The Bronx.

The northern limit of the Twenty-fourth Ward and this annexed district is about 15 miles from the New York City Hall.

The State census of 1875 is said to have been particularly well taken.

There was no New York State census taken in 1885.

The United States census of June 1, 1890, was considered incomplete by local authorities and a police census was taken September 1, 1890.

A State census was taken in 1892, which is said to have been inaccurately compiled; the figures for New York City were summarized by Assembly Districts instead of by wards.

TABLE NO. 39.

Actual Increase in Population of Brooklyn by Wards.

(Data mainly from report of Board of Health for 189°. Unpublished copy.)

Group.	Ward.	Date of Formation.	From what Taken.	United States Census June 1, 1870.	Per Cent. Increase in 3 years.	New York State Census June 1, 1875.	Per Cent. Increase in 5 years.	United States Census June 1, 1880.	Per Cent. Increase in 10 years.	United States Census June 1, 1890.	Per Cent. Increase in 2 years.	New York State Census Feb. 1, 1892.	Estimated Population June 1, 1900.
A	First	.....	.....	6,476	..	16,084	..	18,729	..	20,040	..	22,784	.....
	Second	.....	.....	9,117	..	8,860	..	9,254	..	8,986	..	10,529	.....
	Third	1878	Tenth Ward.	9,984	..	15,809	..	18,271	..	18,754	..	24,140	.....
	Fourth	.....	.....	12,087	..	12,616	..	12,819	..	12,324	..	15,580	.....
	Fifth	.....	.....	20,490	..	18,591	..	18,517	..	20,175	..	19,175	.....
	Sixth	.....	.....	28,296	..	34,072	..	35,437	..	37,693	..	48,939	.....
	Seventh	.....	.....	(13,312)	..	26,488	..	31,663	..	35,726	..	39,490	.....
	Tenth	.....	.....	34,592	..	24,866	..	27,140	..	34,031	..	50,318	.....
	Eleventh	.....	.....	21,243	..	21,629	..	21,680	..	22,693	..	25,007	.....
	Twelfth	.....	.....	18,302	..	17,525	..	22,201	..	27,368	..	31,734	.....
	Twentieth	.....	.....	19,179	..	21,430	..	24,188	..	24,136	..	26,120	.....
	Total	.....	.....	193,078	12.9	217,070	10.0	239,899	9.2	261,926	19.8	313,816	.....
B	Thirteenth	.....	.....	18,711	..	19,547	..	21,029	..	21,628	..	24,282	.....
	Fourteenth	.....	.....	20,649	..	23,925	..	25,559	..	27,246	..	32,629	.....
	Fifteenth	.....	.....	18,406	..	21,255	..	23,654	..	27,630	..	30,319	.....
	Sixteenth	.....	.....	26,438	..	39,206	..	42,712	..	45,720	..	51,152	.....
	Seventeenth	.....	.....	17,353	..	23,998	..	30,088	..	41,424	..	46,315	.....
	Eighteenth	.....	.....	(8,607)	..	(12,459)	..	(14,926)	..	(21,960)	..	21,267	.....
	Nineteenth	.....	.....	16,321	..	21,908	..	27,661	..	36,244	..	38,287	.....
	Total	.....	.....	126,485	28.3	162,298	14.4	185,629	18.5	221,852	10.5	245,151	.....

C	Eighth	.....	.....	9,592	..	12,127	..	17,388	..	31,239	..	42,758	.....
	Ninth	.....	.....	12,279	..	13,643	..	15,044	..	17,696	..	21,084	.....
	Twenty-first	.....	.....	27,904	..	24,834	..	37,956	..	50,118	..	57,362	.....
	Twenty-second	.....	.....	11,761	..	17,736	..	25,473	..	50,250	..	57,807	.....
	Twenty-third	1873.	Seventh Ward.	(5,000)	..	10,005	..	14,396	..	29,348	..	33,292	.....
	Twenty-fourth	1873.	Ninth Ward.	(3,000)	..	5,799	..	8,823	..	16,771	..	17,888	.....
	Twenty-fifth	1873.	Seventh Ward.	(5,000)	..	13,081	..	19,055	..	44,638	..	56,682	.....
	Twenty-seventh	1892.	Eighteenth Ward.	(2,000)	..	(3,000)	..	(5,000)	..	25,000	..	34,695	.....
	Twenty-eighth	1892.	Eighteenth Ward.	(1,000)	..	(2,000)	..	(4,000)	..	28,000	..	38,892	.....
	Total	.....	.....	77,536	31.8	102,225	38.0	141,135	107.5	293,060	23.0	360,450	.....
D	Twenty-sixth	1886.	New Lots.	9,800	..	11,047	..	13,655	..	29,505	..	38,541	.....
	Twenty-ninth	1894.	Flatbush.	6,309	..	6,940	..	7,634	..	12,338	..	12,625	.....
	Thirtieth	1894.	New Utrecht.	3,296	..	3,843	..	4,742	..	8,854	..	9,129	.....
	Thirty-first	1894.	Gravesend.	2,131	..	2,180	..	3,674	..	6,937	..	8,418	.....
	Thirty-second	1896.	Flatlands.	2,286	..	2,651	..	3,127	..	4,075	..	4,234	.....
	Total	.....	.....	23,822	11.9	26,661	23.1	32,832	88.0	61,709	18.2	72,947	.....
	Grand total	.....	.....	420,921	21.0	509,154	17.7	599,495	40.0	838,547	18.8	996,071	.....

Inmates of institutions, for Kings county not included in ward figures.
Present Third Ward formed from Tenth in 1878. Old Third Ward is now part of the First.
Present Ninth Ward took in part of the Twenty-second Ward in 1891.

In the time at disposal was unable to gain access to a copy of police census of 1890. Was told that one was taken, but records could not be found. From page 65, Brooklyn Health Department, report 1894, it appears that police census gave 48,000 more than U. S. census, an excess of six per cent. This amount has therefore been added in Table 43 to Federal enumeration for 1890, to make results comparable with those for Manhattan and Bronx.

The changes in ward lines in 1873 and 1892, by which the distant and then sparsely settled portion of Ward 7 was cut off to form Wards 23 and 25, introduces a small irregularity in the summary for 1870, by districts. To correct for this we assume the populations of the parts set off to have been as shown by the figures in parentheses and deduct the same from the census figures for Ward 7 before entering in column for 1870. Wards 9, 27 and 28 are similarly treated. The actual census enumerations were, Ward 7, 23,212 in 1870; Ward 9, 15,279 in 1870; Ward 18, 11,607 in 1870; 17,453 in 1875; 23,926 in 1880, and 74,960 in 1890. This, of course, affects only the distribution, the aggregate being precisely the census figures.

It will be noted that the figures above differ slightly from those given on page 50 of Mr. deVarona's report of 1896, on additional supply for Brooklyn, but not enough to materially affect the results of the present study.

TABLE NO. 40.

Actual Increase in Population of Borough of Richmond.

(Data from report of Health Board of New York City for 1898. Unpublished Copy.)

Ward.	When Formed.	Formerly Known as	U.S. Census, 1870.	N.Y. State Census, 1875.	U.S. Census, 1880.	U.S. Census, 1890.	N.Y. State Census, 1892.
First	1898	Castleton	9,504	10,957	12,679	16,423	17,261
Second	"	Middletown	7,589	8,332	9,029	10,557	11,477
Third	"	Northfield	5,949	6,619	7,014	9,811	9,641
Fourth	"	Southfield	5,080	4,426	4,980	6,644	6,324
Fifth	"	Westfield	4,905	4,862	5,289	8,258	8,648
Total	.....	.....	33,029	35,196	38,991	51,693	53,351
Per cent. increase from preceding enumeration			.....	6.6	10.8	32.6	3.2

TABLE NO. 41.

Actual Increase in Population of Borough of Queens.

Ward.	When Formed.	Formerly Known as	U.S. Census, 1870.	N.Y. State Census, 1875.	U.S. Census, 1880.	U.S. Census, 1890.	N.Y. State Census, 1892.
First	1898	* Long Island City	.....	15,587	17,129	30,505	35,745
Second	"	Newtown	20,274	10,614	9,804	17,549	19,776
Third	"	Flushing	14,650	15,357	15,906	19,803	20,816
Fourth	"	Jamaica	7,745	8,983	10,088	14,441	17,654
Fifth	"	** Hempstead	.....	.....	.....	.....	.....
Total	.....	.....	42,669	50,541	52,927	82,299	93,991
Per cent. of increase from preceding enumeration			.....	18.4	4.7	55.5	14.2

* Formed in 1870 from Newtown.

** Only part of Hempstead was consolidated with the other towns forming Greater New York and the population previous to consolidation cannot be given. Assembly Bill 964 in 1898 returned to the old town all of Hempstead taken, except Far Rockaway.

TABLE NO. 44½.

Past and Future Population of the New York Metropolitan District.

(As Compiled and Projected by J. R. Freeman, C. E., Dec., 1899-Jan., 1900, as Basis for Estimates of Future Water Supply Required.)

See Diagram No. 13, near page 88.

The enumerations to 1900, inclusive, are from data given in Tables Nos. 38 to 44.

* The upper line of figures for 1900, for the several boroughs of New York, are the estimates of the Department of Health, made by geometric progression from census of 1895 and that of 1892. See page 290.

** The lower line of figures for 1900, is after an allowance for checking of growth by "hard times."

While I consider the best information points to an increase for New York as a whole during the next three decades of 30 per cent., 25 per cent. and 20 per cent. (see pages 290–3), I have adopted so large a ratio only for Brooklyn and Queens, and, to be conservative, have figured growth for Manhattan and Bronx at 25 per cent., 22½ per cent. and 20 per cent., respectively, for the next three decades.

Richmond's future population is assumed at what appears a probable growth in view of water-front and proximity.

I have assumed that growth in the fourth decade, 1930 to 1940, will be less, say 15 per cent.

Year of Enumeration.	Manhattan.	Bronx.	Manhattan and Bronx.	Brooklyn.	Queens.	Brooklyn and Queens.	Richmond	Total for Greater New York.	New Jersey Towns.		Total Metropolitan District.
									Within 15 miles of New York City Hall.	Within 20 and Outside 15 Miles.	All of Greater New York and New Jersey Towns Within 15 Miles.
1840	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	439,000
1850	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	785,000
1860	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1,360,000
1870	942,292	28,981	971,273	420,921	42,669	463,590	33,029	1,467,892	304,671	66,151	1,771,563
1875	1,005,692	36,194	1,041,886	509,154	50,451	559,695	35,196	1,636,777	380,240	77,365	2,017,017
1880	1,164,673	47,626	1,212,299	599,495	52,927	652,422	38,991	1,903,712	437,756	92,377	2,341,468
1885	.....	.....	.....	.....	.....	.....	.....	.....	523,063	108,575	.....
1890	1,629,460	91,257	1,720,717	886,547	82,239	968,846	51,693	2,741,256	614,497	131,036	3,355,753
1895	1,742,985	125,075	1,868,060	1,091,646	103,970	1,195,616	59,847	3,123,523	734,129	162,085	3,867,652
*1900	1,980,223	168,867	2,149,090	1,249,226	133,366	1,382,592	68,254	3,599,936	880,064	185,940	4,480,000
**1900	.....	.....	2,080,000	.....	.....	1,360,000	70,000	3,510,000	795,000	170,000	4,300,000
1905	.....	.....	2,340,000	.....	.....	1,565,000	75,000	3,980,000	.....	.....	4,836,000
1910	.....	.....	2,600,000	.....	.....	1,770,000	90,000	4,460,000	.....	.....	5,375,000
1915	.....	.....	2,892,000	.....	.....	1,990,000	110,000	4,992,000	.....	.....	5,979,000
1920	.....	.....	3,185,000	.....	.....	2,210,000	130,000	5,525,000	.....	.....	6,584,000
1925	.....	.....	3,503,000	.....	.....	2,430,000	150,000	6,083,000	.....	.....	7,242,000
1930	.....	.....	3,822,000	.....	.....	2,650,000	175,000	6,649,000	.....	.....	7,900,000
1935	.....	.....	4,108,000	.....	.....	2,850,000	195,000	7,153,000	.....	.....	8,492,000
1940	.....	.....	4,395,000	.....	.....	3,047,000	210,000	7,652,000	.....	.....	9,085,000

The estimates of population used in Table No. 5 are from table above, but are given in round numbers.

TABLE No. 42.

Actual Increase in Population of New Jersey Towns Within 15-miles Radius of New York City Hall.

TOWN.	COUNTY.	U. S. Census, 1870.	Per cent. of Increase, 1870–1875.	Census, 1875.	Per cent. of Increase, 1875–1880.	U. S. Census, 1880.	Per cent. of Increase, 1880–1885.	1885.	Per cent. of Increase, 1885–1890.	U. S. Census, 1890.	Per cent. of Increase, 1890–1895.	1895.	Per cent. 1895–1900.	1900.
a Palisades	Bergen	.....	....	1,894	....	2,302	....	2,333	....	2,590	....	3,787	....	.....
b Englewood	"	.....	....	3,932	....	4,076	....	4,429	....	4,785	....	5,690	....	.....
c Ridgefield	"	.....	....	3,564	....	3,952	....	4,487	....	5,477	....	4,081	....	.....
d North Barbadoes	"	4,929	....	4,352	....	4,248	....	4,983	....	6,004	....	7,281	....	.....
e Boiling Springs	"	.....	....	.....	....	.....	....	.....	....	1,438	....	.....	....	.....
Saddle River	"	1,168	....	1,424	....	1,355	....	1,584	....	2,197	....	3,662	....	.....
f Union	"	2,057	....	3,057	....	3,164	....	3,914	....	1,560	....	1,852	....	.....
Lodi	"	3,221	....	(3,620)	....	4,071	....	(4,570)	....	5,131	....	(5,750)	....	.....
North Bergen	Hudson	3,032	....	3,928	....	4,268	....	5,459	....	5,715	....	8,427	....	.....
Hoboken	"	20,297	....	24,766	....	30,999	....	37,721	....	43,648	....	54,283	....	.....
West Hoboken	"	4,132	....	5,219	....	5,441	....	7,162	....	11,665	....	18,296	...	.....
Weehawken	"	597	....	602	....	1,101	....	1,469	....	1,943	....	2,577	....	.....
Union (Town and Township)	"	6,737	....	2,580	....	7,159	....	10,179	....	12,770	....	18,341	....	.....
Jersey City	"	82,546	....	109,227	....	120 722	....	153,513	....	163,003	....	182,713	....	.....
Bayonne	"	3,834	....	5,836	....	9,372	....	13,080	....	19,033	....	19,836	....	.....
Kearney	"	974	....	1,401	....	777	....	3,338	....	7,064	....	10,487	....	.....
Harrison	"	4,129	....	(5,330)	....	6,898	....	(7,600)	....	8,338	....	(9,200)	....	.....
Guttenberg	"	.....	....	.....	....	1,206	....	1,615	....	1,947	....	3,626	....	.....
Acquackanonck	Passaic	4,368	...	1,631	....	1,781	....	2,038	....	2,562	....	3,598	....	.....
Passaic City	"	incl. above	...	(4,710)	....	6,532	....	(9,780)	....	13,028	....	(19,500)	....	.....
g Franklin	Essex	.....	....	1,556	....	1,617	....	1,602	....	2,007	....	3,076	....	.....
h Belleville	"	3,644	....	2,795	....	3,004	....	3,285	....	3,487	....	4,563	....	.....
Newark	"	105,059	....	123,310	....	136,508	....	152,988	....	181,830	....	215,805	....	.....
Clinton	"	2,240	....	2,647	....	2,742	....	2,830	....	3,684	....	5,470	....	.....

Orange	"	9,348	....	10,813	....	13,207	....	15,231	....	18,844	....	22,792	....	.....
South Orange	"	2,963	....	3,252	....	6,089	....	4,225	....	8,076	....	5,108	....	.....
East Orange	"	4,315	....	6,497	....	8,349	....	10,328	....	13,282	....	17,927	....	.....
West Orange	"	2,106	....	2,835	....	3,385	....	3,812	....	4,358	....	5,854	....	.....
Bloomfield	"	4,580	....	5,425	....	5,748	....	6,502	....	7,708	....	8,093	....	.....
Montclair	"	2,853	....	4,034	....	5,147	....	6,327	....	8,656	....	11,753	....	.....
Elizabeth	Union	20,832	....	25,923	....	28,229	....	32,119	....	37,764	....	43,834	....	.....
i Union	"	2,314	....	2,313	....	2,418	....	2,589	....	2,846	...	3,412	....	.....
j Linden	"	1,396	....	1,757	....	1,889	....	1,971	....	2,057	....	2,428	....	.....
Total		303,671	25.3	380,240	15.2	437,756	19.2	523,063	17.5	614,497	19.5	734,129	....	.....

a Formed in 1871, from part of Hackensack.
b Formed in 1871, from part of Hackensack.
c Formed in 1871, from part of Hackensack.
d Included in Hackensack village but co-extensive in 1880. In 1871, part of Midland.
e Organized from part of Union township
f Part taken to form Boiling Springs in 1889. Rutherford borough formerly in Union. Incorporated in 1880.
g In 1874, from part of Belleville.
h In 1871, part of Woodville annexed. In 1874, part to Franklin. Woodville not in census of 1875.
i In 1871, part to Crawford.
In 1871, part to Crawford.
Figures in parentheses are interpolated.

TABLE No. 43.

Increase in Population of New Jersey Towns, Mainly Within 20-Mile Radius of New York City Hall, but Outside a 15-Mile Radius.

TOWN.	COUNTY.	U. S. Census, 1870.	Per cent. of Increase, 1870–1875.	1875.	Per cent. of Increase, 1875–1880.	U. S. Census, 1880.	Per cent. of Increase, 1880–1885.	1885.	Per cent. of Increase, 1885–1890.	U. S. Census, 1890.	Per cent. of Increase, 1890–1895.	1895.	Per cent. of Increase, 1895–1900.	1900.
Harrington	Bergen	2,664	....	2,676	....	2,570	....	2,604	....	2,769	....	3,250	....	.....
a Washington	"	2,514	....	3,000	....	2,853	....	2,714	....	2,942	....	1,259	....	.....
b Midland	"	.....	....	1,620	....	1,591	....	1,617	....	1,829	....	2,638	....	.....
c Ridgewood	"	.....	....	.....	....	1,478	....	1,776	....	1,841	....	2,969	....	.....
Paterson	Passaic	33,579	....	38,814	....	51,031	....	63,273	....	78,347	....	97,344	....	.....
Manchester	"	1,156	....	1,344	....	1,513	....	1,639	....	2,576	....	4,993	....	.....
Little Falls	"	1,282	...	1,457	....	1,404	....	1,707	....	1,890	....	2,419	....	.....
Wayne	"	1,521	....	1,630	....	1,757	....	1,866	....	2,004	....	2,099	....	.....
Caldwell	Essex	2,727	....	2,897	....	3,167	....	3,336	....	3,638	...	2,642	....	.....
Livingston	"	1,157	....	1,200	....	1,401	....	1,275	....	1,197	....	1,311	....	.....
Millburn	"	1,675	....	1,541	....	1,743	....	2,022	....	2,437	....	2,762	....	.....
d Springfield	Union	770	....	810	....	844	....	847	....	959	....	930	....	.....
				1,489										
Summit	"	1,176	....		....	1,910	....	2,539	....	3,502	....	4,450	....	.....
e Westfield	"	2,753	....	3,026	....	2,216	....	2,352	....	2,739	....	3,713	....	.....
f Cranford	"	.....	....	1,110	....	1,184	....	1,251	....	1,717	....	2,145	....	.....
g Clark	"	331	....	329	....	353	....	363	....	367	....	384	....	.....
Rahway	"	6,258	....	6,947	....	6,455	....	6,861	....	7,105	....	7,945	....	.....
Woodbridge	Middlesex	3,717	....	3,705	....	4,099	....	4,227	....	4,665	....	5,802	....	.....
Perth Amboy	"	2,861	....	3,770	....	4,808	....	6,311	....	9,512	....	13,030	....	.....
Total	.....	66,151	16.9	77,365	19.5	92,377	17.5	108,575	20.7	131,036	23.6	162,085	....	.....

a Orril township organized from parts of Hoboken and Washington township in 1885.
In 1885 Orril township not given, but Hoboken 2,898 population.
In 1895 Orril township 2,249, and Hoboken 2,377 population.
b In 1871 from part of New Barbadoes.
c In 1876 from part of Franklin. In 1875 Franklin had 3,300 population.
d In 1871 part of Cranford.
e In 1871 part of Cranford.
f In 1871 from parts of Clark, Linden, Springfield, Union and Westfield.
g In 1871 part to Cranford.

CONCLUSIONS. Past Rate of Increase. TABLE NO. 44.

Actual Past Rate of Increase of Population in Greater New York and Vicinity. Total Population at Each Census from 1870 to 1895.

Date of Enumeration.	Manhattan.	Bronx.	Manhattan and Bronx.	Brooklyn.	Queens.	Richmond.	Total for Greater New York.	New Jersey Towns within 15 miles.	New Jersey Towns within 20 miles and outside 15 miles.	Total New York Plus New Jersey Towns within 15 miles.
June 1, 1870	942,292	28,981	971,273	420,921	42,669	33,029	1,467,892	303,671	66,151	1,771,563
June 1, 1875	1,005,692	36,194	1,041,886	509,154	50,541	35,196	1,636,777	380,240	77,365	2,017,017
June 1, 1880	1,164,673	41,626	1,212,299	599,495	52,927	38,991	1,903,712	437,756	92,377	2,341,468
June 1, 1885	.....	.....	.....	.....	.....	.....	.....	523,063	108,575	.....
†June 1, 1890	1,629,460	81,257	Pol.	Pol.	U.S.	U.S.	*2,741,256	U.S.	U.S.	
February 1, 1892	.....	.....	1,801,739	996,071	93,991	53,351	2,945,152	(660,000)	.....	(3,625,000)
April, 1895	1,742,985	108,075	1,868,660	(1,091,646)	(103,970)	(59,847)	(3,123,523)	734,129	162,085	(3,867,652)
January 1, 1900 (Estimated)	1,980,223	168,867	2,149,090	(1,249,226)	(133,366)	(68,254)	(3,599,936)	(880,064)	(185,940)	(4,480,000)
Per Cent. Increase Per 10 Years.										
10 years, 1870–1880	23.6	64.0	25.0	42.5	24.0	18.1	29.7	44.2	39.7	32.2
10 years, 1880–1890	39.9	91.5	42.0	47.8	55.4	32.6	44.0	40.3	41.9	43.3
5 years, 1890–1895	7.0	37.0	8.6	23.1	26.3	15.8	14.0	19.5	23.7	(15.2)
10 years, 1890–1900 (assumed per cent)	21.5	85.0	24.8	41.0	62.1	32.1	31.3	43.3	41.9	(33.6)
Per Cent. of Increase Per Decade.										
15 years, 1880–1895	30.	83.	32.	47.	61.	32.	38.	40.	44.	38.
20 years, 1875–1895	32.	84.	34.	46.	48.	30.	38.	38.	45.	38.
25 years, 1870–1895	27.	76.	29.	45.	46.	26.	35.	41.	42.	36.

For the 15-year period 1880–1895, also for the 20-year period 1875–1895, and the 25-year period 1870–1895, the increase per decade is computed in geometrical ratio for 10-year increments.

Figures in parentheses are based mainly upon interpolation.

The above estimates cover the population within the present territorial limits; the New York Health Department in preparing the official estimate base this on the State census of 1895 and the United States census of 1880, and interpolate and extend in geometrical progression. The Brooklyn estimate is based upon State census of February, 1892, with natural increase in geometrical progression.

* Total for Greater New York in 1900 is obtained by using police census for Manhattan and Bronx and Brooklyn, and the United States census for the other municipal divisions.

Diagram No. 9 is drawn from data above.

† For June 1, 1890, the police census taken in September is used for Manhattan and The Bronx and Brooklyn, but the figures given above are understood to have been reduced by proportion to allow for increase since June 1.

Appendix No. 5.

BASIS FOR ESTIMATE OF COST OF NEW AQUEDUCTS.

In considering the relative cost and economy of the ten or more different sources from which it is possible to procure an additional supply of water for New York we had occasion to make estimates of the carrying capacity and cost of aqueducts, tunnels and steel conduits and siphons aggregating nearly five hundred miles. For some of these lines a close estimate was desired and for others a rough approximation would serve. To judge of the relative economy of the various projects it was important that these estimates should all rest upon the same basis. The cost per mile of a pipe or aqueduct of a given carrying capacity varies largely according to the slope or fall per mile that can be given to it. For example, an aqueduct of 500 million gallons per day capacity having a slope of 2 feet per mile will carry only 353 million gallons per day if given a slope of 1 foot per mile, and will cost substantially the same in either case. It was, therefore, desirable to have a ready means of computing the size and slope best adapted to a given location.

The following diagrams and tables were prepared with much care after going over a part of the lines and noting those characteristics of the ground which affect the cost of work, and are intended to give the cost *under average conditions.* For especially difficult ground a correction is to be added, and deduction may be made for easy ground.

The greater part of the cost in work of this kind on aqueducts and tunnels goes out in labor, and if the price per day for common labor were fixed at the same per diem and for the same length of working hours as contemplated by certain municipal ordinances for work inside the city limits, the cost of much of this work would be increased about 30 per cent. I have estimated common labor at the actual price now commonly paid on large contract work of this character. The unit prices in these following estimates are somewhat larger than the prices at which the similar work of the Massachusetts Metropolitan Water Board (the most recent large example of work of this class) has actually been executed, enough so, it is believed, to fully cover all recent advances in cost of labor and materials.

I am much indebted to the kindness of F. P. Stearns, Chief Engineer of the Massachusetts Metropolitan Water Board, and to Thomas W. Richardson, Engineer of its Dam and Aqueduct Department,

who arranged the leave of absence for Mr. Horace Ropes, Assistant Engineer, at that time engaged upon the final location surveys for the Weston Aqueduct, so that he was able to assist me in the field, in making a hasty reconnaissance of various possible aqueduct lines east of the Hudson, and subsequently across the Hudson and along the Wallkill and Esopus Creeks; comparing the character of the country from mile to mile in roughness and probable cost of grading with that in which he had been supervising aqueduct construction and surveys for the past four or five years. Prior to this aqueduct construction, Mr. Ropes had had much experience in the West in railroad construction and in the reconnaissance for railroad location. Mr. Ropes subsequently prepared the larger part of the data which I have summarized in pages 312 to 317, which data were based largely upon his personal experience gained as Assistant Engineer in charge of the construction of the two miles of tunnel forming a part of the 300 million gallon Wachusett Aqueduct of the Massachusetts Metropolitan Supply, and his subsequent supervision of a section of the cut-and-cover masonry aqueduct forming a part of the same system.

This Wachusett Tunnel was built in 1896 and 1897 and is thus the most recent and, next to the New Croton, is the largest water supply tunnel yet constructed. Its Chief Engineer was Mr. F. P. Stearns, who had previously had experience in difficult tunneling while Assistant Engineer on the Dorchester Bay Tunnel for the Boston improved sewerage, completed in 1883. Mr. A. Fteley, Chief Engineer, Croton Aqueduct Commissioners, under whom the new Croton Tunnel was completed, acted as Consulting Engineer. We may, therefore, accept the experience gained upon this Wachusett Tunnel as a prudent guide, with confidence that it had the benefit of the latest developments in the art of tunneling.

The cut-and-cover portion of the Wachusett Aqueduct was designed by Mr. F. P. Stearns, who had the advantage of serving under Mr. Joseph P. Davis and Mr. A. Fteley, as an Assistant Engineer on the construction of the Sudbury Aqueduct, twenty-three years ago, and who gave a great deal of study to perfecting the details of this Wachusett design, so that it should contain the utmost economy of material, while its masonry arch and abutments possess ample factors of strength and stability against loads from within or from without, whether empty or filled with water, covered with earth or uncovered. It is without doubt the finest example of masonry aqueduct construction yet to be found.

We were, therefore, very fortunate while making these estimates of costs of aqueducts for New York to have the assistance of the engineer under

whose immediate supervision this entire tunnel and a part of this cut-and-cover aqueduct for the new Boston works was constructed.

That the kind of structure proposed may be more readily understood by those not familiar with aqueduct construction, Figures 56 to 59 are appended.

I have followed this general design of Mr. Stearns in the sections of Figs. Nos. 60, 61, 62, 63, 64, 65, in which Section A is substantially the same throughout as the Wachusett, both for cut-and-cover aqueduct and tunnel. Sections B and C are drawn in same general proportion as A, and without having had time for a complete analysis of the stresses under all conditions, it is believed the quantities of material assumed in the estimates of Table No. 47 are ample for the present purposes of preliminary estimate.

PHOTOGRAPHS EXHIBITING THE STRUCTURAL FEATURES OF THE PRINCIPAL TYPES OF AQUEDUCT UPON WHICH THESE ESTIMATES OF COST ARE BASED.

Fig. 56.

REPRESENTATIVE SECTION ILLUSTRATING BRICK-LINED TUNNEL IN UNSOUND ROCK.

(Wachusett Aqueduct, Boston New Metropolitan Supply.)

This is of but little less diameter than the proposed 500-million gallon tunnels from East Branch to Rye and Park Hill.

Fig. 57.

REPRESENTATIVE SECTION OF UNLINED TUNNEL IN SOUND ROCK.

(Wachusett Aqueduct, Boston New Metropolitan Supply.)

This is of nearly the same diameter as the half-size or 500-million gallon tunnels on Housatonic Project. (Note the roughness of side walls as affecting the hydraulic coefficient of roughness.)

Similar photographs of interior of the Washington, D. C., aqueduct tunnel illustrate the reason for the high value of the hydraulic factor "n" for unlined tunnels, adopted in the discussion of the "Side-lined Tunnel."

PHOTOGRAPHS EXHIBITING THE STRUCTURAL FEATURES OF THE PRINCIPAL TYPES OF AQUEDUCT UPON WHICH THESE ESTIMATES OF COST ARE BASED.

Fig. 58.

REPRESENTATIVE SECTION OF OPEN CHANNEL.

Side Slopes, 1 vertical inch, 3 horizontal; Bed and Slopes Covered with Gravel.

The open channel shown above is of substantially the same type and size proposed for conveying the water of the Ten Mile river from Pawling to Sodom Reservoir.

Fig. 59.

STANDARD SECTION OF "CUT AND COVER" AQUEDUCT.

Proposed in this report for additional supply of water to New York.

These photographs are from the Wachusett Aqueduct, constructed in 1896–97 by The Massachusetts Metropolitan Water Board, which is of exactly the same dimensions as "Section Size A," Tables 45 to 47, used as a basis in estimating cost of certain aqueducts herein proposed.

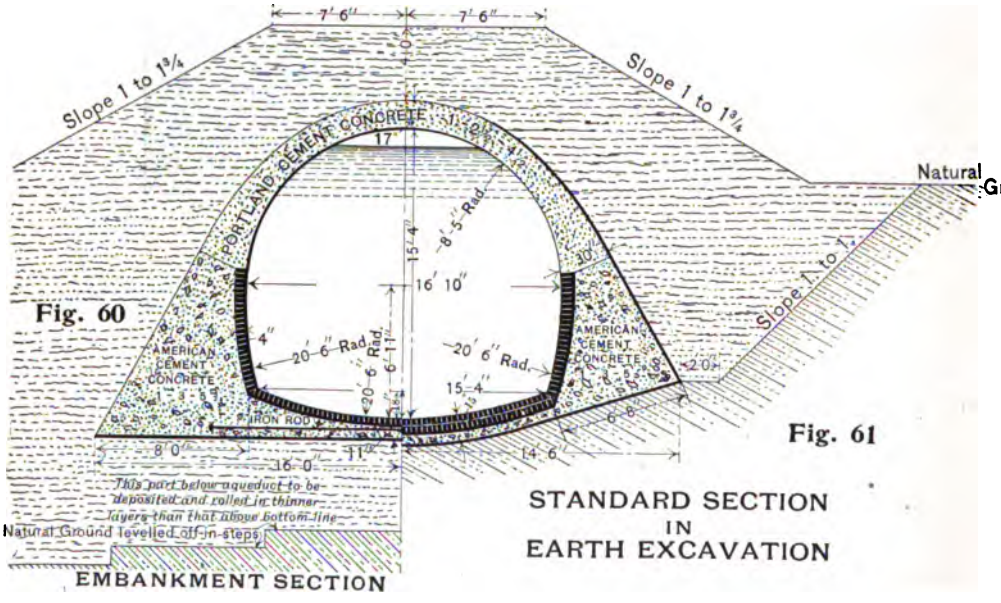


Fig. 60

Fig. 61

STANDARD SECTION
IN
EARTH EXCAVATION

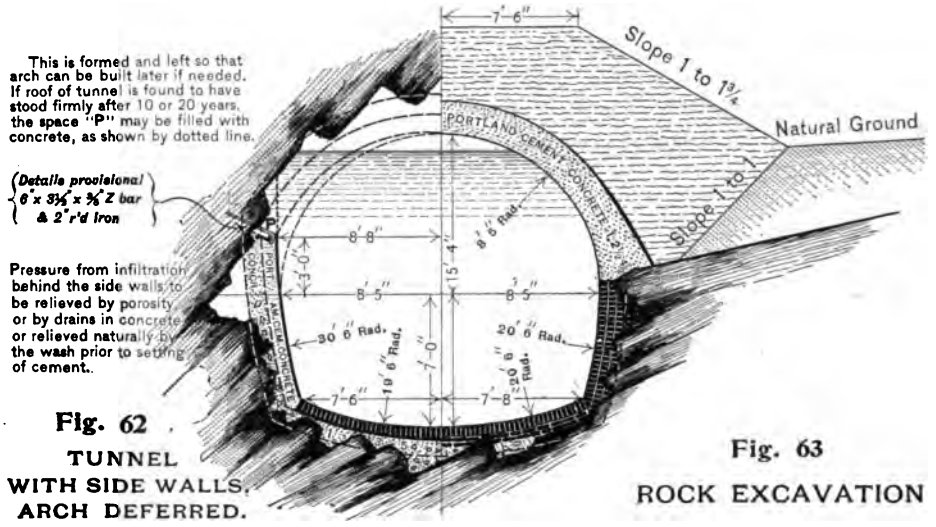

This is formed and left so that arch can be built later if needed. If roof of tunnel is found to have stood firmly after 10 or 20 years, the space "P" may be filled with concrete, as shown by dotted line.
Detail provisional 6" x 3½" x ⅜" Z bar & 2" r'd Iron
Pressure from infiltration behind the side walls to be relieved by porosity or by drains in concrete or relieved naturally by the wash prior to setting of cement.
7'-6"
Slope 1 to 1¼
Natural Ground
Slope 1 to 1
PORTLAND CEMENT CONCRETE
8'-8"
3'-0"
15'-4"
8'-5"
8'-5"
8'-6" Rad.
30'-6" Rad.
18'-6" Rad.
7'-0"
20'-6" Rad.
20'-6" Rad.
7'-6"
7'-8"

Fig. 62

TUNNEL WITH SIDE WALLS, ARCH DEFERRED.

Fig. 63

ROCK EXCAVATION

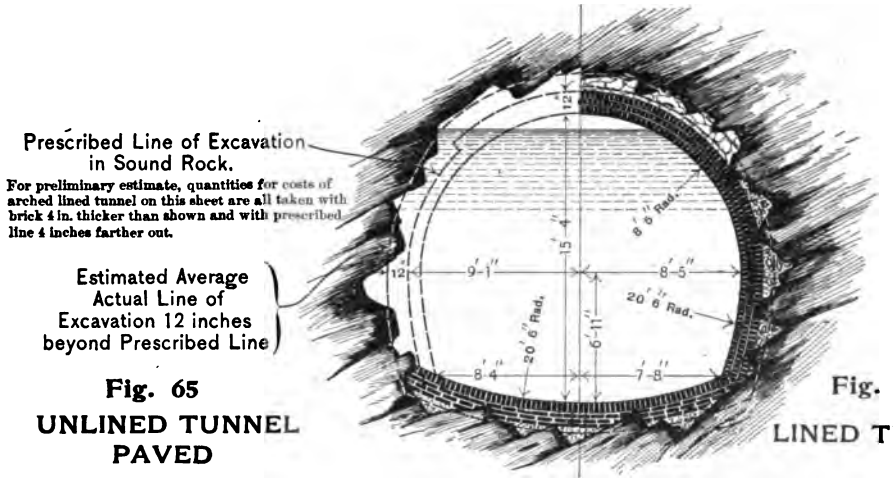

Prescribed Line of Excavation in Sound Rock.
For preliminary estimate, quantities for costs of arched lined tunnel on this sheet are all taken with brick 4 in. thicker than shown and with prescribed line 4 inches farther out.
Estimated Average Actual Line of Excavation 12 inches beyond Prescribed Line
12"
8'-6" Rad.
15'-4"
12"
9'-1"
8'-5"
20'-6" Rad.
6'-11"
20'-6" Rad.
8'-4"
7'-8"

Fig. 65

UNLINED TUNNEL PAVED

Fig. 64

LINED TUNNEL

TABLE No. 45.

LEADING DIMENSIONS OF AQUEDUCT SECTIONS.

Details provisional for purpose of estimate.

The Sectional Drawings represent Section "B" to scale. Sections A and C are nearly proportional thereto. The differences in sizes are shown by table of leading dimensions below. All dimensions in feet unless stated otherwise.

CUT AND COVER AQUEDUCTS.	Section Size A, Same Size as Wachusett.	Section Size B, as Drawn.	Section Size C.
Standard Section in Earth Excavation—			
Centre height, inside	10.50	15.35	20.0
Maximum width, inside	11.50	16.83	22.0
Height of maximum width above invert	4.75	6.93	9.0
Width at edge of invert	10.50	15.35	20.0
Middle ordinate of invert	1.04	1.5	1.8
Height of brick lining above edge of invert	4.4	6.4	8.0
Radius of invert, inside	14.00	20.5	28.0
" walls, inside	14.00	20.5	28.0
" arch, inside	5.75	8.42	11.0
Extreme depth, outside	12.58	18.0	23.0
" width at bottom	20.00	29.0	37.0
Thickness of concrete wall-bottom	4.50	6.67	8.0
" paved invert	4" brick 7" concrete	8" brick 7" concrete	8" brick 8" concrete
" concrete spring of arch	23 in.	30 in.	36 in.
" concrete crown of arch	14 in.	17 in.	20 in.
Depth of fill on top	4.00	4.00	4.00
Width of top of embankment	14.00	15.00	16.00
Slope of sides of embankment	1¾ to 1	1¾ to 1	1¾ to 1
On Embankment—			
All dimensions same as given above, except :			
Extreme depth, outside =	12.25	17.67	22.67
Thickness of paved invert =	4" brick 7" concrete	4" brick 7" concrete	4" brick 8" concrete
Extreme width at bottom =	23.00	32.00	41.0
Thickness of concrete wall at bottom	6.00	8.00	10.0
In Rock Excavation—	A.	B.	C.
Centre height, inside	10.50	15.35	20 0
Maximum width, inside	11.50	16.83	22.0
Height of maximum width above invert	4.75	6.93	9.0
Width at edge of invert	10.50	15.35	20.0
Middle ordinate of invert	1.04	1.5	1.8
Radius of invert	14.0	20.5	28.0
" walls	14.0	20.5	28.0
" arch	5.75	8.42	11.0
Thickness of concrete at crown	14 in.	17 in	20 in.
Least thickness brick lining	4 in.	4 in.	4 in.
" brick paving	4 in.	4 in.	4 in.
Thickness of concrete at spring of arch	Depends on depth of rock excavation.		

TABLE NO. 45—*Continued.*

TUNNELS.	Section Size A.	Section Size B.	Section Size C.
Lined Tunnel.			
Centre height, inside	10.83	15.35	20.0
Maximum width, inside	12.17	16.83	22.0
Height of maximum width above invert	4.75	6.93	9.0
Width at edge of invert	11.25	15.35	20.0
Middle ordinate of invert	1.17	1.5	1.8
Radius of invert	14.0	20.5	28.0
" walls	14.33	20.5	28.0
" arch	6.08	8.42	11.0
For Purpose of Estimate—			
Least thickness brick paving	8 in.	8 in.	8 in.
" " walls	12 in.	12 in.	12 in.
" " crown	16 in.	16 in.	20 in.
Actual dimensions in sound rock may be 4 inches less, as shown in drawing.			
Excavation for Lined and Unlined Tunnels.			
For Purpose of Estimate—			
Maximum depth, prescribed line of excavation	12.8	17.3	22.6
" width, prescribed line of excavation	14.2	18.9	24.0
Width at invert, prescribed line of excavation	13.2	17.3	22.0
In sound rock may be 8 inches less, as shown in drawing.			
Tunnel with Side Lining, Used only in Sound Rock.	A.	B.	C.
Excavation figured as per dimensions in drawing.			
Height of wall above invert	6.75	10.0	13.0
Width at top of wall, inside	12.5	17.33	22.7
" middle	12.17	16.83	22.0
Height of middle line above invert	4.75	7.0	9.0
Width at edge of invert, inside	11.0	15.0	19.5
Middle ordinate of invert	1.17	1.5	1.8
Radius of invert	13.0	19.5	27.0
" lower side walls	22.5	30.5	38.0
Least thickness of brick paving	4 in.	4 in.	4 in.
Average thickness concrete, estmd., under paving	4 in.	4 in.	4 in.
Least thickness concrete side walls	8 in.	8 in.	8 in.
Average thickness concrete, estmd., in side walls	20 in.	20 in.	20 in.
Maximum depth, prescribed excavation	12.2	16.7	21.3
" width, prescribed excavation	13.8	18.7	24.0
Width at invert, prescribed excavation	12.3	16.3	20.8
" middle prescribed excavation	13.5	18.2	23.3

To fit certain conditions, the grade in the tunnel portion of the Wachusett Aqueduct was made lighter than that of the "Cut and Cover" portion; and then to compensate for the decreased velocity due to the decrease in grade, as well as for that due to the greater roughness, fair to be expected, in the brick lining of tunnels, the width in the clear of the lined section was made 8 inches greater, and the depth inside 4 inches greater than for the corresponding "Cut and Cover" Aqueduct, thereby making the tunnel section nearly equivalent in hydraulic capacity to "Cut and Cover" work on the same gradient.

The same finished inside dimensions have been followed above in Section "A" as for the Wachusett Aqueduct. For Sections "B" and "C" the dimensions of the finished inside of the lined arched tunnel are made the same as for the "Cut and Cover" Aqueduct, for convenience in comparing the hydra ulic factors.

TABLE No. 46.

HYDRAULIC ELEMENTS OF AQUEDUCT AND TUNNEL SECTIONS.

"Cut and Cover" Masonry Aqueduct. (Dimensions provisional for purposes of estimate.) The sectional drawings represent Section B to scale.	Section of Size A, Same Size as Wachusett.	Section of Size B, Same as Drawn.	Section of Size C, Proportional to Section B.
Mean of maximum width and depth, feet	11.0	16.1	21.0
Total area of section, inside of lining, square feet	100.8	214.	366.
Diameter of circle of equal area, feet	11.33	16.5	21.6
Depth of water for maximum delivery (about), feet	9.95	14.5	18.9
Area of section at depth of maximum delivery (about), square feet	99.0	211.	358.
Diameter of full circle of equal hydraulic capacity, with same smoothness of walls, feet	11.5	16.8	21.9
Hydraulic mean radius at depth of maximum delivery	3.17	4.64	6.02
Hydraulic factor "n" in Kutter formula (assumed like Sudbury)	.0125	.0125	.0125
Corresponding coefficient "C" with slope 1 foot per mile	145.	152.	157.
Mean velocity, with slope 1.0 foot per mile, f. p. s.	3.55	4.50	5.30
Flow in million gallons per 24 hours at maximum depth, slope 0.50 per mile	161.	434.	865.
" " " 0.80 "	203.	549.	1095.
" " " 1.00 "	226.	614.	1220.
" " " 1.50 "	278.	752.	1500.
" " " 2.00 "	321.	866.	1720.
" " " 2.50 "	358.	968.	1920.
" " " 3.00 "	392.	1060.	2100.
" " " 4.00 "	458.	1220.	2415.

Lined Masonry Tunnel. Sections B and C of same size of finished waterway as "Cut and Cover" Aqueduct. For equal capacity under same slope, with same depth, the width needs to be made about 7 per cent. greater than for "Cut and Cover" construction, to allow for effect of greater roughness of lining upon flow.	Tunnel A is Larger Than "Cut and Cover." Same as Wachusett. A.	B.	C.
Mean of maximum width and depth, feet	11.50	16.1	21.0
Total area of section, inside of lining, square feet	108.	214.	366.
Diameter of circle of equal area, feet	11.7	16.5	21.6
Depth of water for maximum delivery taken same as "C. and C." aqueduct, about, feet	9.95	14.5	18.9
Area of section at depth of maximum delivery, about, square feet	104.	211.	358.
Diameter of full circle of equal hydraulic capacity, with same smoothness	11.9	16.8	21.9
Hydraulic mean radius at depth of maximum delivery	3.28	4.64	6.02
Hydraulic factor "n" in Kutter formula (assumed like New Croton)	.0140	.0140	.0140
Corresponding coefficient "C," with slope 1 foot per mile	132.	138.	142.
Mean velocity, with slope 1 foot per mile, f. p. s.	3.29	4.09	4.79
Flow in million gallons per 24 hours at max. depth above, slope 0.50 per mile	157.	395.	783.
" " " 0.80 "	199.	498.	990.
" " " 1.00 "	222.	557.	1105.
" " " 1.50 "	272.	683.	1355.
" " " 2.00 "	316.	790.	1560.
" " " 2.50 "	350.	880.	1740.
" " " 3.00 "	383.	960.	1910.
" " " 4.00 "	440.	1110.	2210.

TABLE NO. 46—*Continued.*

HYDRAULIC ELEMENTS FOR TUNNEL SECTIONS.

	Section of Size A.	Section of Size B.	Section of Size C.
Unlined Tunnel. (Bottom Paved.)			
Of same size in excavation as for lined tunnel in sound rock. Dimensions for hydraulic purposes estimated to half-way between prescribed line and actual rock wall, equivalent to six inches outside of prescribed line for sound rock, as shown on drawing.			
Depth of water for max.del. taken same as "C. and C." Aqueduct (about) ft.	9.95	14.5	18.9
Area of section at depth of maximum delivery (about), square feet	128.	241.	411.
Hydraulic mean radius at depth of maximum delivery	3.89	5.16	6.77
Hydraulic factor "n" in Kutter formula, derived from Sudbury and Wachusett unlined tunnels when only half filled, and corrected to allow for probable value at full depth	.028	.028	.028
Corresponding coefficient "C," with slope one foot per mile	68.	73.	76.
Mean velocity for slope of one foot per mile, f. p. s.	1.85	2.28	2.74
Flow in million gallons per 24 hours at maximum depth, slope 0.50 per mile	109.	252.	515.
" " " 0.80 "	138.	318.	650.
" " " 1.00 "	150.	355.	725.
" " " 1.50 "	185.	435.	890.
" " " 2.00 "	215.	500.	1030.
" " " 2.50 "	240.	560.	1150.
" " " 3.00 "	260.	620.	1260.
" " " 4.00 "	300.	710.	1460.
Tunnel with Side Lining.			
Bottom paved, no arch in top.			
Area of wet section to top of wall, square feet	73.	156.	262.
Hydraulic mean radius at this depth	3.24	4.78	6.16
Hydraulic factor "n" in Kutter formula, assumed larger than for brick lining to allow for possible extra roughness of concrete	.015	.015	.015
Corresponding coefficient "C," with slope one foot per mile	122.	129.	133.
Flow in million gallons per 24 hours, slope one foot per mile	142.	391.	769.
Ratio to flow of "Cut and Cover" Aqueduct at maximum depth	.623	.633	.626
Depth for maximum delivery assumed same as lined tunnel	9.95	14.5	18.9
Area of section at depth of maximum delivery, square feet	110.	232.	405.
Hydraulic mean radius at this depth	3.74	5.42	7.09
	With care to keep concrete centres smooth and knocking off any bad projections above P these quantities could probably be made 10% greater or same as for Cut and Cover.		
Hydraulic actor "n" in Kutter formula (with space P filled)	.0170	.0170	.0170
Corresponding coefficient "C," with slope one foot per mile	110.	117.	121.
Mean velocity for slope of 1.0 foot per mile, f. p. s.	2.93	3.75	4.42
Flow in million gals. per 24 hours at max. depth assumed, slope 1.0 per mile	210.	560.	1160.
" " " 1.5 "	260.	690.	1420.
" " " 2.0 "	300.	800.	1650.
" " " 2.5 "	330.	890.	1840.
" " " 3.0 "	365.	975.	2020.
" " " 4.0 "	420.	1120.	2320.

CAPACITY AND COST

Under Average Conditions. Subject to Modification, According to Special Structures,

(For Preliminary Estimates in Reconnaissance, N. Y. W. S.,

TABLE

COSTS (PER

Estimated Cost of Grading for

BASIS—1st. A computation of quantities and costs at fair average prices for the "Ideal Section"; all excavation in dry earth on level ground; the depth of cut such that back-filling balances excavation.

2d. An analysis by Horace Ropes, C. E., of the ratio of *actual* quantities and costs found upon representative sections of the Wachusett (Boston) Aqueduct to those for the "Ideal Section."

3d. A hasty reconnaissance by Horace Ropes, C. E., Assistant Engineer on Wachusett

Actual unit prices per contract on the Wachusett Aqueduct were lower than these herein assumed.	Section 4. An "Easy Section."			Section 5. An "Easy Section."		
	Approx. per cent. of Length.	Character.	Average Depth, Cut or Fill.	Approx. per cent. of Length.	Character.	Average Depth, Cut or Fill.
Approximate lengths, class and approximate depths.....	50	Earth	10 feet.	30	Earth	10 feet.
	20	Rock	8 "	20	Rock	half
	8	Embankment	3 "	10	Earth	23 feet.
	22	Earth	20 "	40	"	17 "
Actual quantities figured at fair average prices, as given—	Cubic Yards.		Cost.	Cubic Yards.		Cost.
Earth excavation, at $0.35 per cubic yard	13.77		$4 82	13.203		$4 62
Earth borrow, at 0.30 "	4.43		1 33	5.572		1 67
Rock excavation, 1.50 "	0.45		68	0.572		0 86
Actual cost of grading per linear foot of aqueduct.....	....		$6 83	....		$7 15
Ratio to cost of "Ideal Section".....	....		1 798	....		1 88

A further study was also made by Mr. Ropes with scale drawings of Section "B" with various assumed depths and side hill angles.

CONCLUSION.—For a general average of conditions found along the lines of proposed "Cut and Cover" Aqueducts from Sodom reservoir to proposed terminal "High Pressure" reservoir at Park Hill, also for average conditions

OF LARGE AQUEDUCTS.

Roughness of Country, Costs Exclusive of Land Damages, Shafts, Surveys and Supervision.

by J. R. FREEMAN, C. E. December, 1899–February, 1900.)

NO. 47.

LINEAR FOOT).

"Cut and Cover" Aqueducts.

Aqueduct Construction, over a large part of the proposed lines from Sodom Reservoir to New York City limits, and over representative portions of lines northerly from Croton Lake to Poughkeepsie, etc., for purpose of comparing roughness of country and probable quality and cost of excavation with those with which he was familiar along the Wachusett and Weston Aqueduct Lines.

Taking four representative sections of Wachusett Aqueduct, Mr. Ropes found from the "final estimates" for paying contractors, in comparison with the "Ideal Section" of earth excavation, 9.5 feet deep, as follows :

Section 6a. Rather Heavy Earth Work.			Section 6b. Heavy Rock Work.			Section 7. Heaviest Earth Work.			Section 10. Long Deep Cut.		
Approx. per cent. of Length.	Character.	Average Depth, Cut or Fill.	Approx. per cent of Length.	Character.	Average Depth, Cut or Fill.	Approx. per cent. of Length.	Character.	Average Depth, Cut or Fill.	Approx. per cent. of Length.	Character.	Average Depth, Cut or Fill.
50	Earth	10 feet.	35	Earth	10 feet.	20	Earth	10 feet.	10	Earth	10 feet.
30	Embankment	10 "	40	Rock	8 "	30	"	16 "	80	"	16 "
20	Earth	22 "	25	"	12 "	10	"	24 "	10	Rock	6 "
						10	"	30 "			
						10	Embankment	10 "			
						20	Rock	8 "			
Cubic Yards.		Cost.	Cubic Yards.		Cost.	Cubic Yards.		Cost.	Cubic Yards.		Cost.
11.15		$3 90	15.01		$5 25	....		....	22.98		$8 04
18.99		5 70	1.21		36	....		....	1.326		40
0.055		0 08	3.19		4 79	....		....	0.90		1 35
....		$9 68	....		$10 40	....		....	....		$9 75
....		2 60	....		2 74	....		....	....		2 575

from Croton Lake northerly to Poughkeepsie, and subject to additions for siphons, extraordinary embankments and bridges, the cost of grading is taken at three times the cost of grading the "Ideal Section."

Cost of excavating "Ideal Section": A (Wachusett)—$3.80 x 3=$11.40; B—$5.35 x 3=$16.00; C—$7.35 x 3= $22.00 per linear foot.

CAPACITY AND COST

Under Average Conditions. Subjects to Modifications, According to Special Structures,

(For Preliminary Estimates in Reconnaissance, N. Y. W. S.,

TABLE

Cost of Masonry for

		Section A.	
		Cu. Yds.	Cost.
In dry earth	Portland concrete arch, at ..... $6 per cubic yard	0.911	$5 47
	American concrete walls and bed, at ..... 5 "	1.324	6 62
	Brick lining, at ..... 13 "	0.249	3 24
	Total estimated cost	.....	$15 33
In wet earth	Portland concrete arch, at ..... $6 per cubic yard	0.911	$5 47
	American concrete walls and bed, at ..... 5 "	1.70	8 50
	Brick lining, at ..... 13 "	0.387	5 03
	Lumber	.....	0 76
	Total estimated cost	.....	$19 76
On embankment	Portland concrete arch, at ..... $6 per cubic yard	0.911	$5 47
	American concrete walls and bed, at ..... 5 "	2.329	11 65
	Brick lining, at ..... 13 "	0.249	3 24
	Tie rods ..... (say)	.....	30
	Total estimated cost	.....	$20 66
In dry rock cut to half depth	Portland concrete arch, at ..... $6 per cubic yard	0.911	$5 47
	American concrete walls and bed, at ..... 5 "	0.966	4 83
	Brick lining, at ..... 13 "	0.249	3 24
	Total estimated cost	.....	$13 54
In wet rock cut to half depth	Portland concrete arch, at ..... $6 per cubic yard	0.911	$5 47
	American concrete walls and bed, at ..... 5 "	1.43	7 15
	Brick lining, at ..... 13 "	3.87	5 03
	Total estimated cost	.....	$17 65
Mean estimated cost adopted			$16 67
Total cost, average conditions, including excavation and masonry			$28 07

Examples for comparison:

SUDBURY AQUEDUCT.—Built in 1876–7, "Cut and Cover" type, "Horseshoe" form.

Maximum width, 9.0 feet; maximum height, 7.7 feet; diameter of circle of equal area, 8.5 feet; diameter of circle of equal hyd. capacity, 8.75 feet.

Delivery by experiment when clean, with slope 1.0 foot per mile, = 109 million gallons in 24 hours.

Average contract cost of 11 sections, covering about 11 miles, excluding sections containing tunnels and special structures, not including cost of land or supervision, $23.86 per linear foot = $126,000 per mile.

OF LARGE AQUEDUCTS.

Roughness of Country, Costs Exclusive of Land Damages, Shafts, Surveys and Supervision.

by J. R. FREEMAN, C. E. December, 1899–February, 1900.

NO. 47.—*Continued.*

"Cut and Cover" Aqueducts.

Section B.		Section C.		Remarks.
Cu. Yds.	Cost.	Cu. Yds.	Cost.	
1.624	$9 74	2.548	$15 29	
2.477	12 39	4.160	20 80	
0.358	4 65	0.464	6 03	
.....	$26 78	.....	$42 12	
1.624	$9 74	2.548	$15 29	
2.690	13 45	4.255	21 28	
0.556	7 23	0.724	9 41	
.....	1 11	.....	1 52	
.....	$31 53	.....	$47 50	
1.624	$9 74	2.548	$15 29	
4.113	20 57	6.530	32 65	
0.358	4 65	0.464	6 03	
.....	40	.....	60	
.....	$35 36	.....	$54 57	
1.624	$9 74	2.548	$15 29	NOTE.—Experience shows that in the ordinary rock cut to half depth the saving in masonry very nearly offsets the extra cost of excavation of rock over earth.
.....	.....	.....	.....	
1.120	14 56	1.340	17 42	
.....	$24 30	.....	$32 71	
1.624	$9 74	2.548	$15 29	
.....	.....	.....	.....	
1.318	17 13	1.600	20 80	
.....	$26 87	.....	$36 09	
$27 82		$40 35		Relative lengths of different kinds adopted for average country traversed in this reconnaissance: Dry earth, 20 per cent.; wet earth, 20 per cent.; dry rock, 30 per cent.; wet rock, 20 per cent.; embankment, 10 per cent.
$43 87		$62 40		Per mile: A—$148,000; B—$232,000; C—$330,000.

WACHUSETT AQUEDUCT.—Built in 1896–7, "Cut and Cover," type, same as Section "A" above, 6 miles long.

Actual cost for total of all contract work, exclusive of land, preliminary work and supervision, about $24 per foot = $127,000 per mile, or 16 per cent. less than per estimate above.

The actual cost of preliminary work, supervision and land, with some day work in the final preparation for use, averaged about $5 per linear foot, or about $26,000 per mile.

CAPACITY AND COST

Costs Under Average Conditions, Subject to Modification, According

Shafts, Surveys

(Prepared for Preliminary Estimates in Reconnaissance for New York Water Supply, by

TABLE No. 47.—

ESTIMATED COST OF TUNNELS

Lined Tunnel, Sections "B" and "C" of same Size

			Section of Size A.	
			Cu. Yds.	Cost.
Quantities and estimated costs of rock excavation and masonry per linear foot, including compressor plant, pumping, hoisting and contractors' profits.	Actual "heading" excavation, at	$7 00 per c. y.	3.2	$22 40
	" "bench" " at	4 00 "	4.0	16 00
	Brick lining (est. walls 12", arch 16"-16"-20")	13 00 "	1.50	19 50
	" backing, at	6 50 "	0.25	1 62
	Rubble backing (probably), at	4 00 "	0.40	1 60
	Dry packing (probably), at	2 00 "	1.00	2 00
Total estimated costs, exclusive of shafts and supervision				$63 12

Unlined Tunnel with Bottom Paved, of same

		Section of Size A.	
		Cu. Yds.	Cost.
Estimated cost of excavation, as above		....	$38 40
Brick paving on invert (8"), at	$13 00 per cubic yard	0.32	4 16
" backing " at	6 50 "	0.20	1 30
Total cost, exclusive of shafts, supervision and land damage			$43 86

Tunnel with Side Lining, with Bottom Paved, with no Arch in Top, but left in Shape so that be Run at 60 per cent.

		Section of Size A.	
		Cu. Yds.	Cost.
Estimated cost of excavation—Heading, at	$7 00 per cubic yard	3.2	$22 40
" Bench, at	4 00 "	4.25	17 00
Brick paving on invert (4"), at	13 00 "	0.16	2 08
Concrete backing under invert, at	6 50 "	0.30	1 95
American concrete in side lining, at	6 50 "	0.434	2 82
Portland " at	8 00 "	0.370	2 96
Iron work and drill holes		.....	95
Total estimated cost per linear foot, exclusive of shafts and supervision			$50 16

Examples for Comparison:

NEW CROTON AQUEDUCT.—Built in 1882–50, Lined Masonry Tunnel, "Horseshoe" form.

Maximum width, 13.60 feet; maximum height, 13.53 feet; diameter circle of equal area, 14 feet; diameter circle of equal hyd. capacity, 14.18 feet.

Delivery when clean, with slope 0.70 per mile, = 302 million gallons per 24 hours. Same section when clean with slope 1.0 foot per mile, should deliver 412 million gallons per 24 hours.

OF LARGE TUNNELS.

to Quality of Rock for Tunneling, Costs Exclusive of Land Damages, and Supervision.

J. R. FREEMAN, December, 1899, Mainly from Data Furnished by HORACE ROPES, C. E.)

Continued.

IN FAIRLY SOUND ROCK.

of Finished Waterway as "Cut and Cover" Aqueducts.

Section of Size B.		Section of Size C.		Remarks.
Cu. Yds.	Cost.	Cu. Yds.	Cost.	
3.2	$22 40	3.2	$22 40	The corresponding contract prices per cubic yard with only the prescribed section paid for must average from 30 per cent. to 40 per cent. greater than these for contractor to recover cost of over-run.
9.1	36 40	16.1	64 40	
2.31	30 03	3.20	41 60	
0.30	1 95	0.39	2 54	Cost of backing assumed to be fixed by contract at one-half price of lining to six inches outside of prescribed line to induce care in restricting excavation beyond the prescribed line.
0.50	2 00	0.64	2 56	
1.00	2 00	1.25	2 50	
$94 78		$136 00		Per mile: A—$333,000; B—$500,000; C—$718,000.

Size in Excavation as for the Lined Tunnel.

Section of Size B.		Section of Size C.		Remarks.
Cu. Yds.	Cost.	Cu. Yds.	Cost.	
....	$58 80	....	$86 80	
0.50	6 50	0.64	8 32	
0.30	1 95	0.39	2 54	
$67 25		$97 66		Per mile: A—$232,000; B—$355,000; C—$516,000.

the Arch can be Added, if Necessary, in 10 or 20 Years hence, meanwhile the Aqueduct to full Capacity or Less.

Section of Size B.		Section of Size C.		Remarks.
Cu. Yds.	Cost.	Cu. Yds.	Cost.	
3.2	$22 40	3.2	$22 40	
9.4	38 60	10.4	65 60	
0.25	3 25	0.32	4 15	
0.50	3 25	0.64	4 16	
0.680	4 42	0.866	5 64	
0.620	4 96	0.744	5 95	
.....	95	.....	95	
$77 83		$108 85		Per mile: A—$256,000; B—$412,000; C—$575,000.

See Report, 1895, p. 67. Cost horseshoe sections complete, including shafts, 70,415 feet = 13.34 miles, averaged $104.24 per linear foot = $550,000 per mile. Contract prices excluding land and supervision, or 31 per cent. more than for same capacity per above estimate, which includes shafts, etc.

WACHUSETT TUNNEL.—Completed in 1897. Same as Section "A" above. Two miles long (half lined and half unlined).

Actual total cost for all contract work (including shafts) averaged about $33 per foot = about $174,000 per mile, or 38 per cent. less than per estimate above; but the contractor is supposed to have lost money.

In the tunnel lined with masonry I have ventured to adopt for these preliminary estimates slightly larger quantities in rock excavation and a somewhat greater average thickness for the brick lining than are given in Mr. Ropes' estimates, fearing lest the rock along the lines now to be considered might, on the whole, prove less favorable than was found in the large tunnel completed two years ago under his supervision; and also bearing in mind the high cost of the New Croton Tunnel, constructed 1882–1890. Mr. Ropes' prices per cubic yard for the tunnel excavation and its masonry lining are materially higher than the actual contract prices for the Wachusett Tunnel.

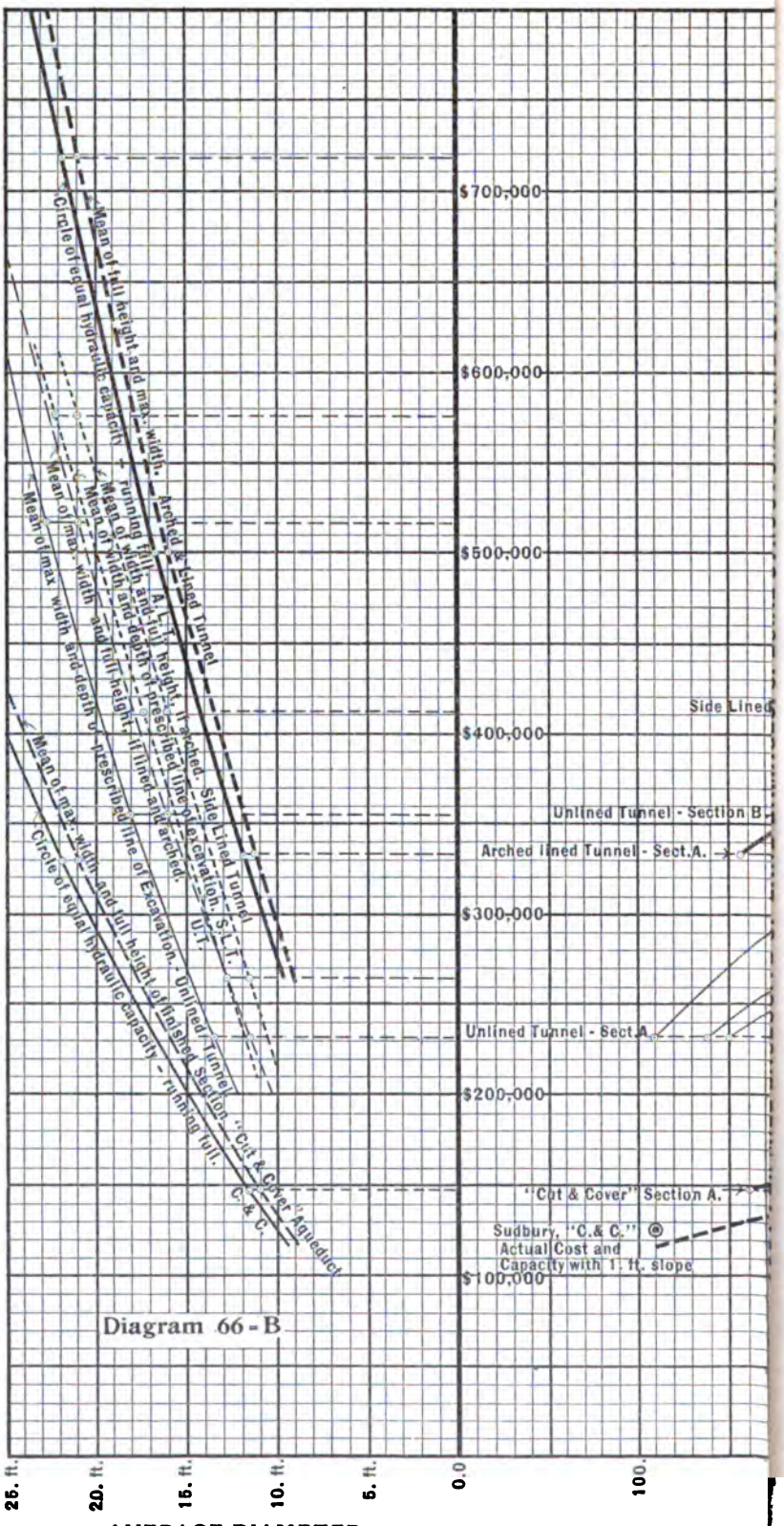
Side-Lined Tunnel.

I have included an estimate for a new type of tunnel section for sound rock in which only the sides and bottom are lined and in which the arch is deferred or omitted, and have followed this type in the estimate of cost of aqueduct lines for those localities where the rock is firm enough to stand without lining in preference to the unlined tunnel, for two reasons:

1st. Because its waterway can be more quickly and thoroughly cleaned from organic growths. The importance of this has been much impressed upon me by what I have learned of the collection of organic matter that is subject to decay, in the upper portion of the Old Croton Aqueduct and the inference that a similar deposit now exists in the upper portion of the New Croton.

2d. Because of the very great frictional resistance to the flow of water found in the unlined portion of the Sudbury and Wachusett Tunnels, notwithstanding the floor of each of these tunnels is smoothly paved. This increased resistance due to the eddies and swirls caused by the projections and irregular faces of the rough rock sides of the tunnel lessens the carrying capacity so much in comparison with that of a lined tunnel, that the economy of omitting the lining is more apparent than real. This use in sound rock of a side-lining without an arch, merely for the purpose of smoothing up the wetted perimeter, is uncommon if not novel, and therefore, requires some further explanation.

In a section of the size of the Wachusett Tunnel, or Section A, Tables 45, 46 and 47, although adding this side lining to the unlined paved section, increases the cost about $6.30 per lineal foot, or $33,000 per mile, and at the same time lessens the area of the waterway about 14 per cent., all avail-


$700,000
$600,000
$500,000
$400,000
$300,000
$200,000
$100,000
Side Lined
Unlined Tunnel - Section B
Arched lined Tunnel - Sect.A.
Unlined Tunnel - Sect.A.
"Cut & Cover" Section A.
Sudbury, "C.& C." Actual Cost and Capacity with 1 ft. slope
Mean of full height and max. width.
Arched & Lined Tunnel
Circle of equal hydraulic capacity - running full.
A. L. T.
Mean of width and full height.
Arched. Side Lined Tunnel
Mean of width and depth of prescribed line of excavation.
S. L. T.
Mean of max. width and full height, if lined and arched.
U.T.
Mean of max. width and depth of prescribed line of Excavation. Unlined Tunnel
Circle of equal hydraulic capacity - running full.
Mean of max. width and full height of finished section.
"Cut & Cover" Aqueduct
C. & C.
Diagram 66 - B
25. ft.
20. ft.
15. ft.
10. ft.
5. ft.
0.0
100.

AVERAGE DIAMETER
Mean of Max. Width and Max. Full Depth.

able hydraulic data concur in leading us to believe that the smoothing up of the side walls would increase the water carrying capacity under a given gradient or slope in a much greater degree, making it equal in carrying capacity to the lined masonry tunnel; so that in effect the cost of the side-lined tunnel for a given capacity would be the same as for the unlined tunnel, while the side-lined tunnel would present the advantage of being much more easily cleaned.

Diagram No. 66 shows that the side-lined tunnel to carry 250 million gallons per day with a fall of 2.0 feet per mile, would cost $248,000 per mile, and have a diameter of excavation to the "prescribed line" averaging 12.25 feet, while the unlined tunnel to be of equal capacity must have 2.0 feet greater diameter of excavation to the "prescribed line" and would cost about $249,000 per mile.

It can be seen by a study of Diagram No. 66, that with the larger sectional areas required by lower grades or larger volumes, the introduction of the side lining leads to a positive saving in cost, when both tunnels—side-lined and unlined—are made of such relative size as to convey equal quantities of water on the same slope. For example, if 500 million gallons were to be brought in with a slope of one foot per mile, the unlined tunnel under average conditions would cost about $425,000 per mile. The side-lined tunnel would under the same average conditions cost about $390,000 per mile, and thus save about $35,000 per mile, besides being more easily kept clean. The average diameter of prescribed excavation for this unlined tunnel must be about 20.35 feet. The average diameter of prescribed excavation for this side-lined tunnel need be only about 16.70 feet. It may at first sight appear paradoxical that a section of 14 per cent. less area will carry 40 per cent. more water merely because of the smoothness of its lining, but during my service under Hiram F. Mills, C. E., I had opportunity to see a similar question put to the proof in the lining of certain long wooden flumes, in which, under his direction, the friction loss was greatly diminished and the capacity increased while reducing the diameter by a 2-inch smooth plank sheathing placed inside the yokes or frames which were built inside the original planking of the flumes. While we have no experimental data on the hydraulic factors for an unlined tunnel flowing full, I believe that the factor of roughness would be found more, rather than less, than the figure which I have adopted and, on the other hand, it should not be difficult to obtain a smoother surface for the concrete side lining than that allowed for in the value adopted for the hydraulic factor n in Table No. 45. Possibly because

of this side lining in a hydraulic tunnel being novel, some question may be presented regarding the tendency of infiltration water to collect behind it, but from discussing this matter with an engineer experienced in tunnel and aqueduct work I feel reassured that these isolated leaks would take care of themselves, before the cement had set, by washing out enough of the finely ground particles of cement from between the particles of sand and broken stone to afford a safe passage for the infiltration, while the centres would retain the coarser particles. At well-defined leaks, drain channels or weepers would be, of course, provided.

It will be noted that while having a minimum thickness of only 8 inches, the average thickness of these side linings is about 20 inches, which with such adhesion of the cement to the rock faces as could be secured by a little care, would, it is believed, give ample stability; but to be on the safe side, the cost of the cap irons and heavy studs shown in the section Fig. 62 are added.

The details of all these quantities, unit-prices and hydraulic factors are presented with fulness in Tables 45, 46 and 47, in order that the basis of these estimates can easily be verified by any engineer familiar with this class of work.

Large Steel Conduits—Construction, Cost and Water-Carrying Capacity.

In nearly all of these aqueduct lines it has appeared expedient to make use of more or less large steel pipe for siphons in crossing valleys. Certain of the possible supplies for Greater New York, for example, that from Esopus Creek, proposed by the Ramapo Company, can be brought in only by means of long lines of steel pipe conduits similar to those through which the East Jersey Water Company brings the Poquannock water to Newark and Jersey City, but, of course, of much larger diameter.

To bring the estimates for the several plans to a common basis, the following tables were prepared and used, with such additions as appeared proper for the extra roughness of country, the special structures or the short length of any particular line.

The reliability, utility and economy under certain conditions, of properly constructed large, steel pipe conduits are now thoroughly established. The worst thing that can be said against them is their cost, which for the large capacities of 250, 500 and 750 million gallons per day, that we have to consider, is more than double the cost of a masonry aqueduct in "cut and cover."

DIAGRAMS

USED AS

Basis of Estimate for the Various Conduit Lines Considered in Report.

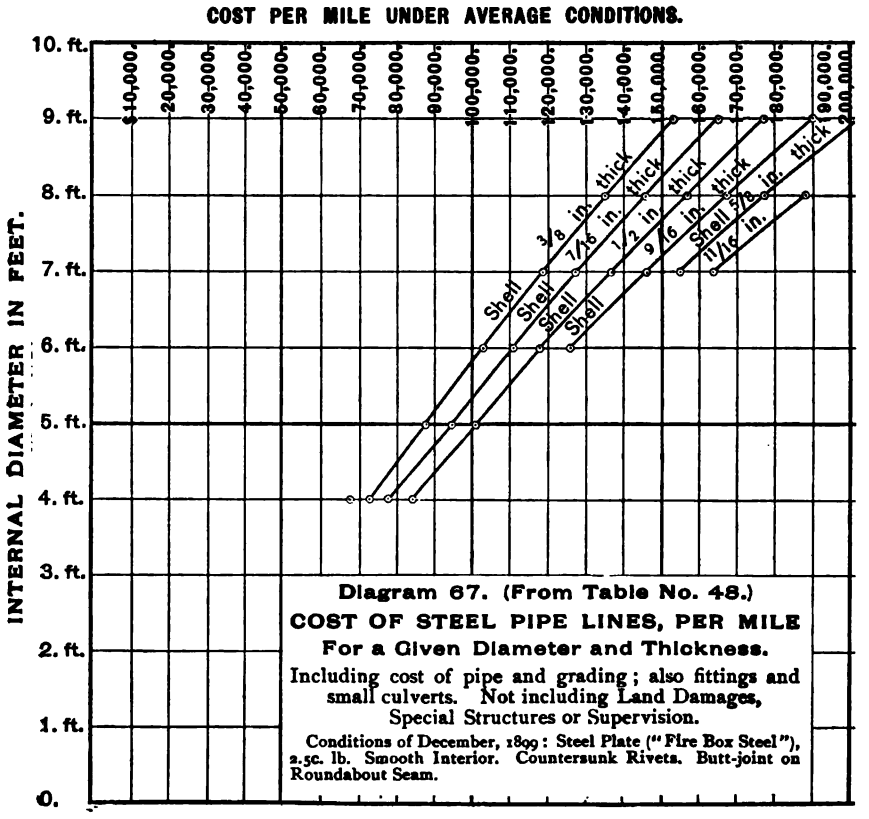


Diagram 67. (From Table No. 48.)

COST OF STEEL PIPE LINES, PER MILE

For a Given Diameter and Thickness.

Including cost of pipe and grading; also fittings and small culverts. Not including Land Damages, Special Structures or Supervision.

Conditions of December, 1899: Steel Plate ("Fire Box Steel"), 2.5c. lb. Smooth Interior. Countersunk Rivets. Butt-joint on Roundabout Seam.

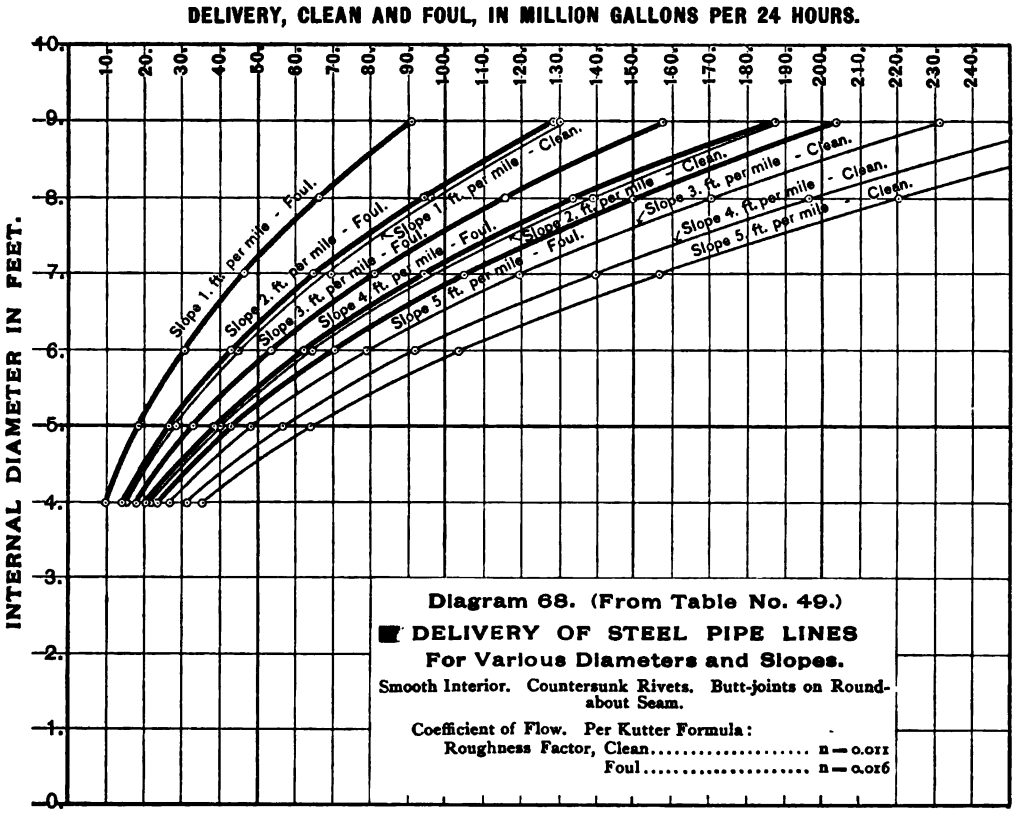

DELIVERY, CLEAN AND FOUL, IN MILLION GALLONS PER 24 HOURS.
INTERNAL DIAMETER IN FEET.
Slope 1. ft. per mile - Foul.
Slope 2. ft. per mile - Foul.
Slope 3. ft. per mile - Foul.
Slope 4. ft. per mile - Foul.
Slope 5. ft. per mile - Foul.
Slope 1. ft. per mile - Clean.
Slope 2. ft. per mile - Clean.
Slope 3. ft. per mile - Clean.
Slope 4. ft. per mile - Clean.
Slope 5. ft. per mile - Clean.

Diagram 68. (From Table No. 49.)

DELIVERY OF STEEL PIPE LINES
For Various Diameters and Slopes.

Smooth Interior. Countersunk Rivets. Butt-joints on Round-about Seam.

Coefficient of Flow. Per Kutter Formula:
Roughness Factor, Clean..... n = 0.011
Foul..... n = 0.016

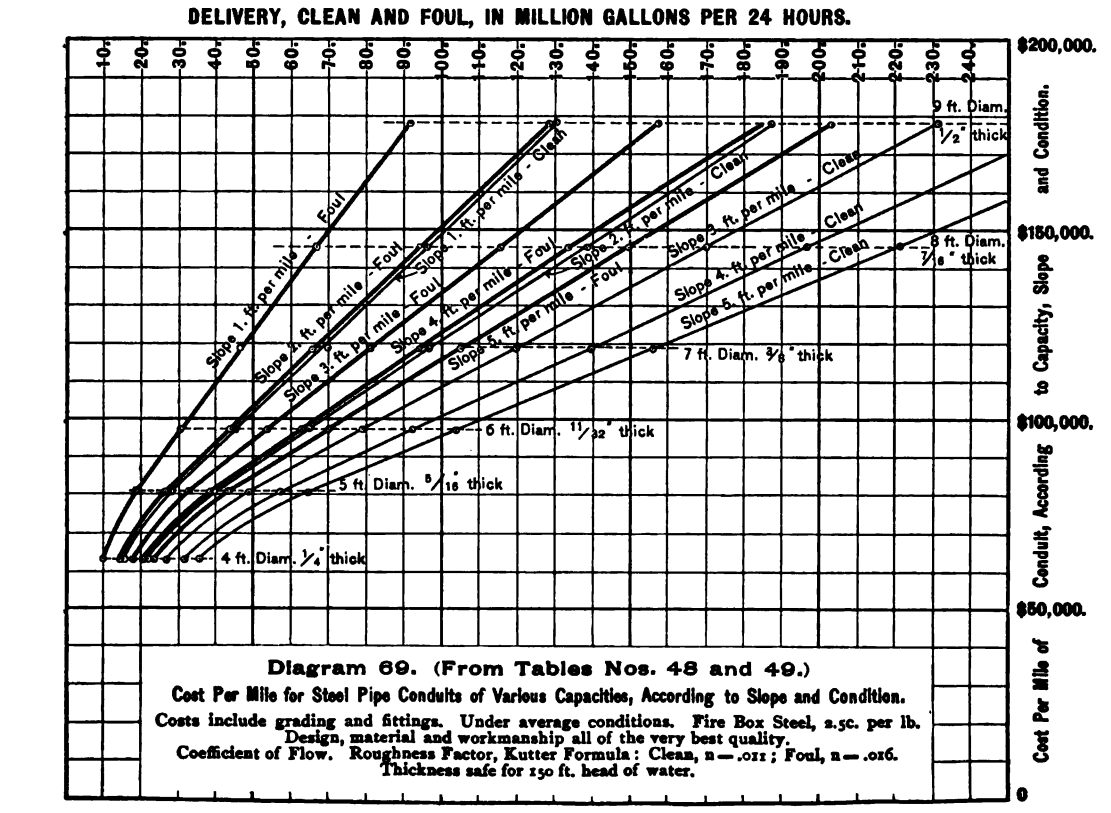
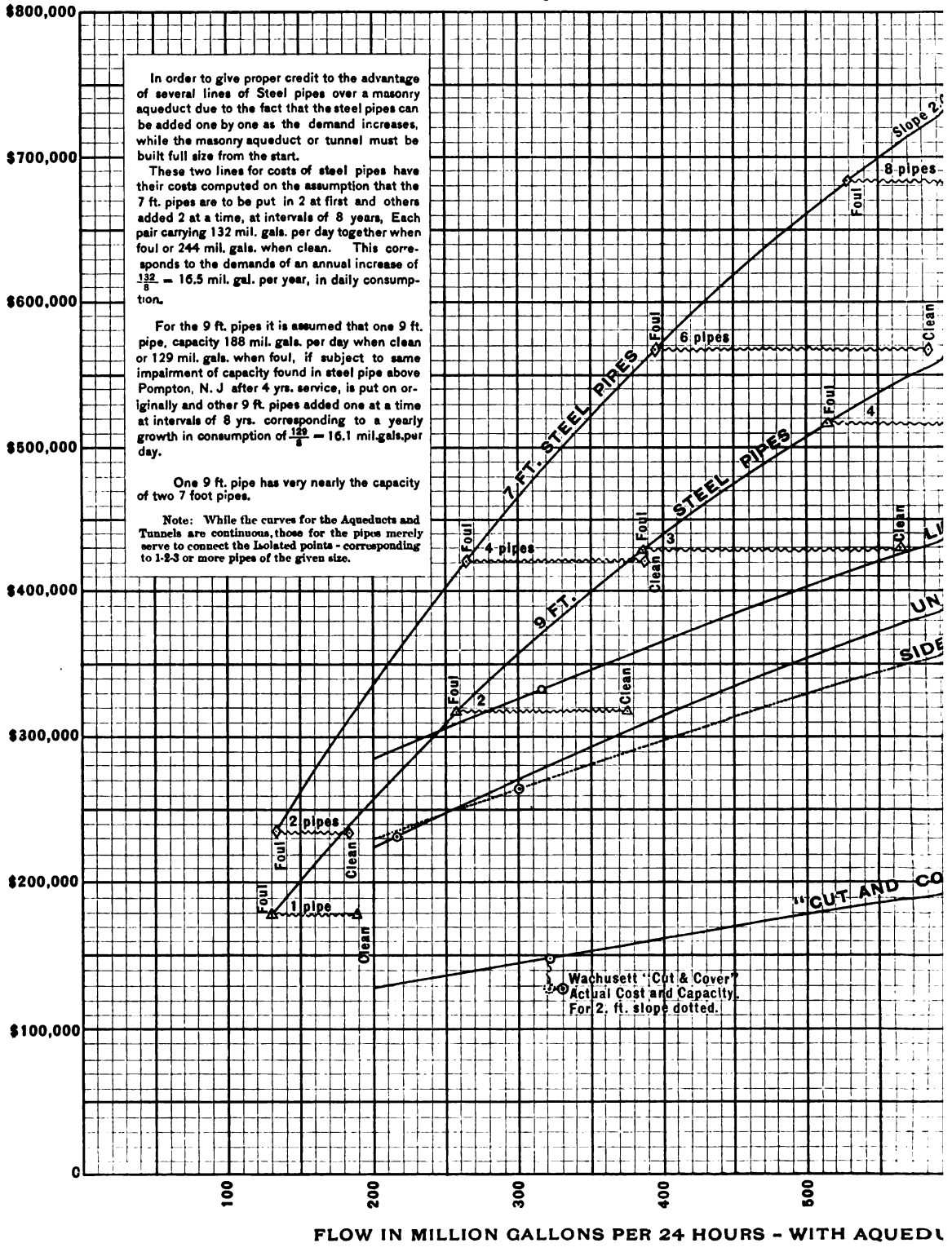

DELIVERY, CLEAN AND FOUL, IN MILLION GALLONS PER 24 HOURS.
Slope 1. ft. per mile - Foul
Slope 2. ft. per mile - Foul
Slope 3. ft. per mile - Foul
Slope 4. ft. per mile - Foul
Slope 5. ft. per mile - Foul
Slope 1. ft. per mile - Clean
Slope 2. ft. per mile - Clean
Slope 3. ft. per mile - Clean
Slope 4. ft. per mile - Clean
Slope 5. ft. per mile - Clean
9 ft. Diam. 1/2" thick
8 ft. Diam. 7/16" thick
7 ft. Diam. 3/8" thick
6 ft. Diam. 11/32" thick
5 ft. Diam. 5/16" thick
4 ft. Diam. 1/4" thick
$200,000.
$150,000.
$100,000.
$50,000.
0
Cost Per Mile of Conduit, According to Capacity, Slope and Condition.

Diagram 69. (From Tables Nos. 48 and 49.)

Cost Per Mile for Steel Pipe Conduits of Various Capacities, According to Slope and Condition.

Costs include grading and fittings. Under average conditions. Fire Box Steel, 2.5c. per lb. Design, material and workmanship all of the very best quality.
Coefficient of Flow. Roughness Factor, Kutter Formula: Clean, n = .011; Foul, n = .016.
Thickness safe for 150 ft. head of water.


$800,000
$700,000
$600,000
$500,000
$400,000
$300,000
$200,000
$100,000
0
100
200
300
400
500
In order to give proper credit to the advantage of several lines of Steel pipes over a masonry aqueduct due to the fact that the steel pipes can be added one by one as the demand increases, while the masonry aqueduct or tunnel must be built full size from the start.
These two lines for costs of steel pipes have their costs computed on the assumption that the 7 ft. pipes are to be put in 2 at first and others added 2 at a time, at intervals of 8 years, Each pair carrying 132 mil. gals. per day together when foul or 244 mil. gals. when clean. This corresponds to the demands of an annual increase of 132/8 = 16.5 mil. gal. per year, in daily consumption.
For the 9 ft. pipes it is assumed that one 9 ft. pipe, capacity 188 mil. gals. per day when clean or 129 mil. gals. when foul, if subject to same impairment of capacity found in steel pipe above Pompton, N. J after 4 yrs. service, is put on originally and other 9 ft. pipes added one at a time at intervals of 8 yrs. corresponding to a yearly growth in consumption of 129/8 = 16.1 mil.gals.per day.
One 9 ft. pipe has very nearly the capacity of two 7 foot pipes.
Note: While the curves for the Aqueducts and Tunnels are continuous, those for the pipes merely serve to connect the Isolated points - corresponding to 1-2-3 or more pipes of the given size.
7 FT. STEEL PIPES
9 FT. STEEL PIPES
Slope 2
8 pipes
6 pipes
4 pipes
2 pipes
1 pipe
Foul
Clean
"CUT AND CO
Wachusett "Cut & Cover" Actual Cost and Capacity. For 2. ft. slope dotted.
FLOW IN MILLION GALLONS PER 24 HOURS - WITH AQUEDU

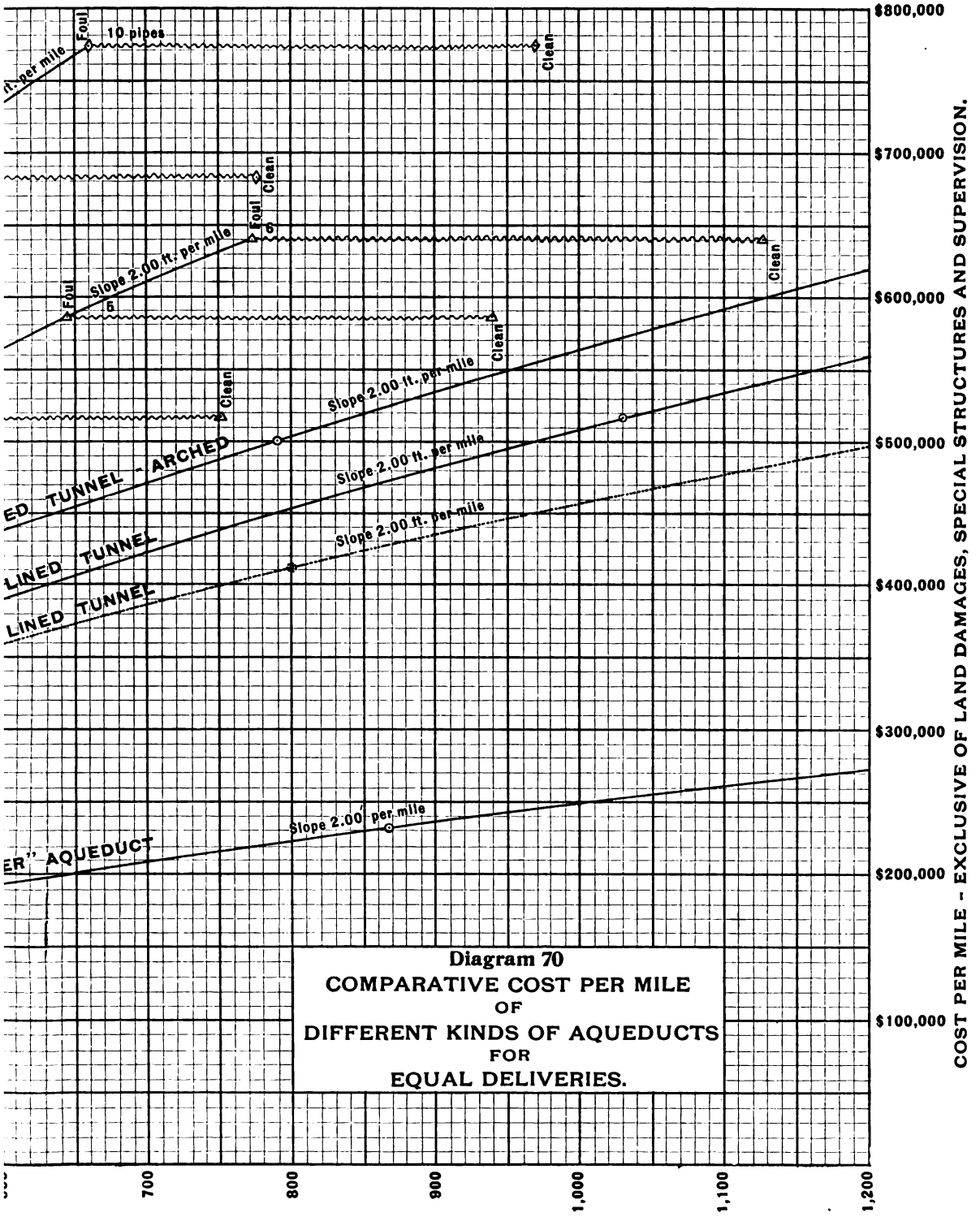


Diagram 70

COMPARATIVE COST PER MILE OF DIFFERENT KINDS OF AQUEDUCTS FOR EQUAL DELIVERIES.

CT CLEAN. AND HYDRAULIC GRADIENT 2.00 FEET PER MILE.

For example—

Steel conduits of capacity sufficient, after the interior has become rough by age or neglect, to convey 200 million gallons per 24 hours, with 2 feet fall per mile, and steel at 2½ cents per pound, would cost, complete in place, for two steel pipes, 8 feet 3 inches diameter, of least allowable thickness (⅜ inch), about	$284,000	per mile
A masonry aqueduct of the very highest type of construction in "cut and cover" of 200 million gallons capacity when foul, or 230 million gallons capacity when clean, 10.1 feet in diameter, would cost about	133,000	"
A lined masonry tunnel of same grade and capacity	298,000	"
Land damages or supervision not being included in either case.		
If one seeks a future supply of 500 million gallons, with 1 foot per mile gradient, and a surplus of 15 per cent. in capacity when new to provide for an impairment like that now existing in the New Croton, the masonry aqueduct in "cut and cover" will cost about	225,000	"
And the fully lined tunnel under average conditions	508,000	"
While with two 9-foot steel pipes at least admissible thickness (⅜ inch) constructed immediately, and three others added successively at eight-year intervals, or five pipes in all, and a delivery of 100 million gallons each in impaired condition, and the cost of each future pipe discounted to present value at 3 per cent. compound interest, would amount to	601,000	"

This great difference in cost is shown on Diagram No. 70.

The effort has therefore been to use as little steel pipe as possible, and to make the largest possible portion of the aqueduct line in "cut-and-cover" masonry, similar to the Wachusett and Sudbury conduits of the Boston supply.

Type of Construction of the Steel Conduits.

The design upon which the following estimates of cost are based is one which I had occasion to work out for another problem five years ago, and while this is a more elaborate and expensive type of construction than followed for the Rochester, East Jersey, Allegheny, Cambridge or New Bedford pipes, I believe the extra life and the additional carrying capacity for a given diameter that can be gained by this design, amply justify its greater cost. In a preliminary estimate resting on such scant surveys as I have had opportunity to make in the present instance, it has moreover appeared prudent to base the unit prices upon that type of construction which has highest first cost, if cheapest in the end.

This conduit should be built of soft, ductile, "fire-box" steel, of the same quality as used for the most difficult situations in high grade steam boilers, a steel with a tensile strength of 60,000 pounds per square inch, an elastic limit of 30,000 pounds per square inch, and of which a specimen strip eight inches long will stretch 25 per cent. in length before rupture, and so soft and tenacious that a plate half an inch thick may be bent back on itself cold and hammered down flat without showing a sign of a crack or fracture of the fold.

It is proposed to proportion the strength against bursting and the strength of the riveted joints on the same basis that is used in designing the best grade of steam boilers; namely, with a factor of safety of 2½ on the elastic limit so that the pressure required to burst the pipe would be about five times the greatest working pressure, and it is proposed to add to the necessary thickness of the steel plate as thus computed, 1/16 inch (or under certain considerations 1-8 inch), as a further allowance for possible thinning of the plates by rust after many years of service. But it is proposed to "rust-proof" this steel conduit inside and out by a thick coating of asphalt put on by immersing each of the 30-foot sections in which the pipe would be made up, in a tank of boiling hot asphalt varnish, or some other approved substance, and to protect this external asphalt coating against abrasion during transportation to the trench and during placing and back-filling—where, when buried, it can never again be reached for re-painting—probably by winding each section of the pipe with creosoted burlap while the asphalt is still hot and adhesive. As a further protection against corrosion, and to make sure of the adhesion of the rust-proof coating to the surface of the steel, and that no rust can start beneath it, it is proposed to pickle all of the plates at the pipe-making shop in very dilute muriatic acid, for the purpose of removing all of the rust and mill scale, precisely as is done in the very best marine boiler work. This pickling is said to cleanse the steel so that it shines almost like silver, and is a process that can be very cheaply applied. This pickling would be done at the field shop immediately before forming and riveting, the plate being well washed when taken from the pickle to remove all traces of acid.*

Life of Steel Conduits.

All these precautions were found to add but a trifle in percentage to the cost, and would make sure of a better, tighter and more durable conduit than heretofore built; one fairly comparable in endurance with a masonry aqueduct, and one which it now appears entirely reasonable to assume would remain in tolerably good condition after even one hundred years of use, if swept out clean and repainted on the inside once in five or ten years. *In estimates of annual depreciation I have nevertheless assumed a life of only fifty years.* This probable life of one hundred years, although it sounds large, appears warranted by the results of an investigation which I had occasion

* The hot asphalt would be applied as soon as possible after forming and riveting, and within as small an interval of time as practicable after the pickling. Possibly recent progress in sand blast work would furnish a better method, and avoid all doubts about a trace of acid being left on the metal. It might develop that while hot asphalt was the best treatment for the outside, painting the inside would secure a smoother surface, and one easier to maintain in the long future. The cost is estimated on the basis of pickling in acid and coating inside and out with asphalt.

to make upon the present condition of certain water-wheel penstocks at Lowell and Lawrence, Mass.

These penstocks were nine feet in diameter and of the same thickness as the thinnest part of the proposed conduit, but they had no such efficient protection against rust originally, and have had no special care in painting since. They are, nevertheless, to-day in good condition after daily use in conveying Merrimack river water for from fifty years to fifty-four years in different instances; and so far as a careful inspection of the surface reveals, bid fair with an occasional painting to do just as good service for the next twenty-five years as they have for the past fifty years.

There may be question if the soft steel of to-day will withstand rust so well as this old wrought iron. Professor Henry M. Howe, now of Columbia University, one of the foremost authorities on steel, investigated this matter for the Massachusetts Metropolitan Water Board in 1895, and concluded that the difference in the relative life against rusting out under these conditions of use, between iron plates like those of which these old penstocks are probably composed, and modern soft steel, is very small. He has recently told me that the developments of the past few years have shown no reason to doubt the equal life of the steel if it is well made and without segregation.

I had occasion at the time above mentioned to inquire particularly regarding the thinning of the plates of iron steamers on the great lakes and elsewhere, and found the statements of naval architects familiar with their endurance, and who had supervised repairs on many boats, confirmed this view of the durability of properly protected iron and of steel plates in fresh water.

Countersunk Rivets and Smooth Joints.

The estimate of cost in Table 48 presupposes the conduit constructed with the inside rivet heads all countersunk like those on the outside of an ocean steamer, and with the roundabout seams made with butt joints smooth on the inside and with an outside cover plate. It is, I believe, almost as important to give a smooth inside to a large iron conduit, as to give a smooth outside to an iron steamboat; and to make the estimate of cost safe, an experienced naval constructor was called upon to aid in estimating the cost of this part of the shop work.

Because of some of the adverse comments made in the discussions of the feasibility of bringing a high pressure gravity supply safely into New York by means of steel pipes 8 feet 3 inches diameter, I am led to state in fairness to those who have proposed doing this, that from the engineer's standpoint these large steel pipes *if designed, built and erected under competent supervision*

from proper specifications, are entirely practicable and as safe as a steel boiler or as a steel railroad bridge. It is merely a question of a proper expenditure of skill and money.

Safety of Large Steel Conduits.

Lest the large diameter of certain of the steel conduit lines proposed in parts of this estimate might excite apprehension among those not familiar with what has heretofore been done with large steel and iron pipes in water-power developments, it may not be amiss to mention that pipes of the same diameter as the largest which enters any of these estimates (9 feet), but of poorer metal, have been in use for fifty years leading to the water-wheels of certain of the mills at Lowell and Lawrence, and the original pipes are still in good condition. It is true that these old pipes were of shorter length and subjected to much less pressure than the proposed conduits, but, as an offset, it is to be noted that they were built of poorer metal and less efficiently protected against corrosion. At Holyoke and at least two other large water-powers, pipes of this same kind up to 12 feet diameter, are in use to-day. Many steel pipes of 6 and 8 feet diameter have been laid in lengths of 800 or 1,000 feet, some exposed to the weather and without expansion joints, and some deeply buried. To the water-power engineer it is a matter of very frequent practice to bury in the ground beneath a large factory a conduit of this size for a feeder to its water-wheels. The mill-yard experience is ample to show that a pipe up to nine and even twelve feet diameter $\times$ $\frac{3}{8}$-inch thick without any stiffening ribs or special support, need not be injured by the back-filling necessary, and will take care of its own temperature strain.

Illustrating the tenacity and ductility of this soft steel, I have photographs of a riveted steel pipe 48 inches in diameter which became thrown out of line in an accident, and was bent at a sharp angle. It yielded by a stretching and buckling of the plate without any rupture in plates or rivets. It was forced back into position without cutting or tearing or opening a joint, and remained water-tight. There are numerous cases where steamships and yachts with hulls constructed of a cheaper grade of soft steel, have gone on to rocks and merely knocked the plates inward a few feet, without tearing open a hole, and I can conceive no accident to the long soft steel shell forming this which could cause a serious rupture; and, indeed, believe that if built under proper supervision a 9-foot steel pipe may be regarded as a safer structure than the 48-inch cast-iron pipes commonly used. The material (firebox steel, as specified), would certainly stretch enough to accommodate the pipe to any slight settlement of an embankment, and it is possible that

even a railroad train derailed on top of the pipe, considering the 5 feet of intervening earth, would not rupture the pipe.

The experience for eight years, summer and winter, with the East Jersey conduit, 21 miles long, 4 feet diameter, of similar metal, and with many riveted wrought-iron pipes in California, as well as with certain large water-wheel feeders already mentioned, shows that these large pipes can safely be left to take care of their own temperature strains without expansion joints, if firmly anchored to ample masonry at their ends.

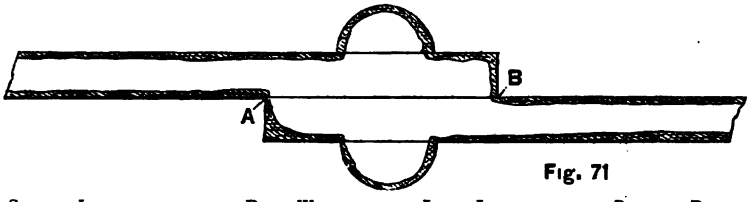


Fig. 71

SKETCH ILLUSTRATING THAT RUST WEAKENS THE JOINT LESS THAN THE BODY OF PLATE.

In considering the effect of rust after long service, it is to be remembered that experience shows that very little rust occurs within the joint between the plates. The joint is the weakest part of a riveted conduit, because of the rivet holes cutting out 30 per cent. of the strength that opposes bursting; but as the conduit becomes weakened as it grows old, through the thinning of its metal by rust, the plates at the joint become weakened only about half as fast as the main body of the plate, because rust only attacks them on one side. I have had recent occasion to make careful observation on some unprotected and badly rusted penstocks that had been more than forty years in use. Many instances of cutting open at the joint showed very little loss at A, B.

In designing the thickness of plates and strength of joint required to withstand the various water pressures, 1/16 inch has been added to the computed thickness to provide against the weakening by rust. If corrosion were to go on with equal rapidity inside and out, then $\frac{1}{8}$ inch could be removed from the plate as a whole, before the factor of safety of $2\frac{1}{2}$ would begin to be encroached upon; in other words, a $\frac{1}{2}$-inch plate could be thinned down to $\frac{3}{8}$-inch.

From observations on various penstocks from twenty-five to fifty years old, it appears reasonable to assume that more than fifty years would clapse before this condition of encroaching upon the factor of safety against bursting, or before the reduction in bursting strength of body of plate would be brought by corrosion below the original estimated strength at the riveted joint.

BASIS FOR ESTIMATES OF WATER-CARRYING CAPACITY OF STEEL CONDUITS.

The following estimates of water-carrying capacity are for steel pipes with rivets countersunk on the inside, and with the plates united on the roundabout seam with butt joints and an outside cover plate. On the longitudinal seam a lap joint is proposed, but the inside heads of the rivets are countersunk.

This gives a more perfect waterway—while new and so long as well cared for—but is a more expensive type of construction than has been followed in the East Jersey, Allegheny, Cambridge, New Bedford or Rochester steel pipes.

The flow and friction loss in the following table is computed by the well-known formula:

$$V = C\sqrt{RI}.$$

In which V = Velocity in feet per second.
R = Mean Hydraulic Radius = one-fourth the diameter.
I = Slope or loss of head per foot in length by friction.
C = Coefficient of flow.

Value of C Adopted in Basis for Estimates.

The only chance for error or difference of opinion regarding the capacity of a given size of conduit under a given slope comes in the value adopted for C.

Although a steel plate painted or properly coated has a surface smoother than the inside surface of a new tar-coated cast-iron water pipe, I have, when inspecting certain large asphalt-coated steel pipe work in progress, seen ample proof that without very careful treatment there is liability that the protective coating, if of asphalt, will harden in ridges. This must reduce the carrying capacity. It is believed this roughness can be prevented by proper care, or by use of a proper process, at a trifling addition to the ordinary cost. This matter should be carefully guarded in the specifications.

To allow for possible increase of roughness with successive painting, the coefficient of frictional resistance for the conduit, when clean, is taken in the above formula, at the same value found in Fitzgerald's experiments upon the 48-inch Rosemary pipe soon after it was cleaned and after about seventeen years of use (see Trans. Am. Soc. C. E., July, 1896, p. 261).

As the most convenient framework now available on which to display and apply results of experiments on friction loss in conduits and by which to make due allowance for differences in diameter and roughness of surface slope, I have taken the " Kutter Diagram " of Hering and Trautwine, based on the well-known Kutter formula (see Plate 8, facing p. 240, of Hering & Trautwine's Translation of Ganguillet & Kutter). The Fitzgerald experiments on this basis give for the factor of roughness a value of n = .011

and the values of the coefficient C, which I have adopted in computing Table No. 49, is substantially that corresponding to a value of n = 0.0110.

I have also confirmed these estimates of velocity for the smooth pipes by the values of the coefficient found in Diagram No. 14 of Hamilton Smith, Jr.'s "Hydraulics."

Impairment of Conduit Capacity.

For a neglected pipe, or a pipe that cannot be shut off and entered and swept out once or twice a year, or for a pipe in which slime or spongilla is likely to thrive, it is assumed that the coefficient of roughness, or *n* in the Kutter formula, will be the same as the value found in the upstream portion of the Hemlock Lake and the East Jersey pipes.

See Herschel's 115 experiments, page 52, for East Jersey pipe.
For 48-inch pipe, with 4 ft. velocity and slope 0.00183..... C = 94.0
This pipe was four years old, obstructed by projecting rivet heads and ridges in asphalt and is supposed to have had organic growths adhering to its interior walls.
Corresponding value of *n* in Kutter formula..... .0162

Also see Herschel's 115 experiments, page 29.

Penstock, 9 feet diameter, at Holyoke, lap joints, projecting rivets, probably free of slime.
With 40 feet velocity, slope .000652..... C = 106.2
Corresponding *n* in Kutter formula..... .0163

Also see Herschel's 115 experiments, page 30.

Hemlock Lake conduit, lap joints, projecting rivets, conduit fifteen years old, measurements by E. Kuichling.
Diameter, 36 in.; velocity, 1.49; slope, .00042 — C 83.0 corresponding *n*..... .0167

Since we assume a pipe without projecting rivet heads, without lap joints, and with ridges and "fingers" of tar, carefully guarded against in the specifications, I have thought it safe to assume that serious neglect and bad accumulations of organic growth would not make the coefficient of roughness greater than n = .016.

It is proposed to sweep the interior as occasion demands, and to repaint the inner surface of these large steel conduits once in about ten years, or whenever rust begins. The cost of sweeping and repainting one mile, including stock, labor and forced ventilation, is estimated at from $1,000 to $1,500.

From personal experience with many water-wheel penstocks, I think it probable that one repainting in ten years would suffice, and from noting how much greater the obstruction of organic growths, spongilla and the like is in the upper five miles as compared with the downstream portions of both the Old and the New Croton Aqueducts, and in the East Jersey pipe above Pompton Notch, I am inclined to believe that with a long line, it would pay to make the diameter for the upper five miles fully 6 inches larger than for the downstream portion.

TABLE No. 48.

Basis of Estimates, Cost of Steel Conduits, *exclusive of Land Damage, Special Structures or Supervision.*

For preliminary estimates—year 1900—under average conditions found on lines over which reconnaissance was made.

Ten per cent. of length assumed to be cut in ledge to same depth as diameter of pipe.

These estimates subject to addition in extra rough or extra rocky territory or to deduction in especially open and easy grading.

Diameter of Pipe. Feet.	Original Thickness of Plate. Inch.	Maximum Working Head of Water. Feet.	Finished Weight per Running Foot with Butt Strap Joint on Round Seams.	Cost of Steel per Foot at 2½ Cents per Pound.	Cost of Shop Manufacture including Rivets, Countersinking, Pickling, Double Asphalt and Burlap Coating, Butt-joints and all Ready to Ship, and Manufacturer's Profit, per Linear Foot.	Transportation, Railroad Freighting from Shop to Trench, Teaming. Per Foot.	Placing in Trench, Caulking and Riveting and Painting Seams. Per Foot.	Excavation, Back Filling and Allowance for Grading and for Ledge. Per Foot.	Total per Foot Allowance for Culverts, Blow-offs, Air-valves, etc.	Total Cost per Pound. Cents.	Total Cost per Linear Foot of Pipe in Trench.	Total Cost per Mile.
9	3/8	112	515	12.87	$7 51	$1 12	$3 00	$4 02	0.65	5.66	$29 15	$154,000
	7/16	135	600	15.00		1 31				5.24	31 45	166,000
	1/2	157	687	17.17		1 50				4.92	33 85	178,000
	9/16	180	773	19.32		1 69				4.67	36 15	191,000
	5/8	202	860	21.50		1 88				4.48	38 55	203,000
8	3/8	126	446	11 15	6 90	91	2 50	3 66	0.60	5.77	25 70	136,000
	7/16	151	520	13.00		1 06				5.34	27 70	146,000
	1/2	177	595	14.87		1 22				5.00	29 70	157,000
	9/16	202	670	16.75		1 37				4.74	31 80	168,000
	5/8	227	744	18.60		1 53				4.54	33 80	178,000
	11/16	253	818	20.45		1 68				4.37	35 80	189,000

7	3/8	144	382	9.55	6 20	74	2 16	3 30	0.55	5.90	22 50	119,000
	7/16	173	445	11.12		87				5.44	24 20	128,000
	1/2	202	510	12.75		99				5.09	25 90	137,000
	9/16	231	574	14.35		1 12				4.82	27 70	146,000
	5/8	260	638	15.95		1 24				4.60	29 40	155,000
	11/16	289	700	17.50		1 36				4.44	31 10	164,000
6	3/8	169	328	8.20	5 40	60	1 91	2 91	0.50	5.94	19 50	103,000
	7/16	202	381	9.52		70				5.50	20 90	111,000
	1/2	236	437	10.92		80				5.14	22 40	118,000
	9/16	270	491	12.27		90				4.86	23 90	126,000
5	3/8	203	276	6.90	4 57	48	1 77	2 55	0.45	5.95	16 70	88,000
	7/16	242	322	8.05		56				5 50	18 00	95,000
	1/2	283	368	9.20		64				5.17	19 20	101,000
4	5/16	215	185	4.65	3 75	28	1 60	2 20	0.40	6.92	12 90	68,000
	3/8	253	223	5.58		34				6.22	13 90	73,000
	7/16	309	260	6.50		40				5.70	14 80	78,000
	1/2	361	298	7.45		46				5.29	15 90	84,000

With two pipes side by side, 3 feet apart, a slight saving in cost of grading is to be allowed for.

Above prices are for work done on a large scale, with field shop near site of work ; under conditions prevailing January 1 to March 1, 1900.

Fire-box steel is estimated in the above at 2½ cents per pound f. o. b. New York. Two years ago this could have been contracted for at about 1½ cents per pound. Good judges engaged in steel manufacture advise that large contracts could probably be placed this year at 2¼ cents if deliveries extend into next year.

TABLE NO. 49.

Basis for Estimates of Water Carrying Capacity of Large Steel Pipe.

For Pipe with Smooth Interior these values for velocity and delivery assume the pipe made with butt joints, and outside Cover Plate, smooth inside, Rivet heads counter-sunk inside, "Rust-proof" coating on interior, substantially free from ridges and wrinkles and originally smooth as in new 48-inch cast-iron pipe of best ordinary commercial quality.

For Pipe with Rough Interior these values assume that in case of long neglect, defective coating, growth of spongilla or other organisms upon interior walls, these pipes may attain substantially the same frictional resistance to flow that was found in East Jersey pipe above Pompton Notch, or in the Rochester conduit and some others.

Diameter of Pipe.	Hydraulic Gradient.		Pipe with Smooth Interior, New and Clean.					Pipe with Rough Interior or Old and Neglected.			
	Feet per Mile.	Slope per 1,000 Feet.	Kutter's C Corresp. to n=.0110	C per Hamilton Smith.	Value of C Used.	Mean Velocity f. p. s.	Million Gallons per 24 Hours.	Kutter's C Corresp. to n=.0160	Value of C Adopted.	Mean Velocity f. p. s.	Million Gallons per 24 Hours.
Pipe of 9.00 feet inside diameter. R=2.25. Area 63.62 square feet	5.00	.9470	159.5	159	159	7.34	501	108.5	108	4.98	304
	4.00	.7576	159	158.5	159	6.57	269	108	108	4.46	188
	3.00	.5682	159	156.5	158	5.65	232	108	108	3.86	158
	2.00	.3788	159	154	157	4.59	188	108	108	3.15	129
	1.00	.1894	158.5	151	155	3.20	131	108	108	2.23	91.5
Pipe 8.25 feet diameter. R=2.06. Area 53.46 (Size proposed by Engineer of Ramapo Co. for Esopus water)	5.00	.9470	158	158	158	6.98	241	107	107	4.73	163
	4.00	.7576	158	157	157	6.20	214	107	107	4.23	146
	3.00	.5682	158	156	157	5.37	185	107	107	3.66	126
	2.00	.3788	157.5	153	155	4.33	150	106.5	106	2.97	99.1
	1.00	.1894	157	148	153	3.04	105	106	106	2.09	72.2
Pipe 8.00 feet inside diameter. R=2.00. Area 50.27 square feet	5.00	.9470	157	157	157	6.83	222	106.5	106	4.62	150
	4.00	.7576	157	156	156	6.07	197	106.5	106	4.13	134
	3.00	.5682	157	155	156	5.26	171	106.5	106	3.58	116
	2.00	.3788	157	153	155	4.27	139	106	106	2.92	94.8
	1.00	.1894	156.5	148	152	2.96	96.1	105.5	105	2.05	66.6

Pipe 7.00 feet inside diameter. R=1 75. Area 38.48 square feet	5.00	.9470	155	154	155	6.31	157	104.5	104	4.23	105
	4.00	.7576	155	153	154	5.61	140	104.5	104	3.79	94.2
	3.00	.5682	154.5	151	153	4.82	120	104	104	3.28	81.5
	2.00	.3788	154	148	151	3.89	96.7	103.5	103	2.65	65.9
	1.00	.1894	153	144	149	2.71	69.9	103	103	1.88	46.7
Pipe 6.00 feet inside diameter. R=1.50. Area 28.27 square feet	5.00	.9470	152	151	152	5.72	104	102	102	3.84	70.2
	4.00	.7576	151.5	148	150	5.06	92.4	101.5	102	3.44	62.9
	3.00	.5682	151	147	149	4.35	79.4	101	101	2.95	53.9
	2.00	.3788	150.5	144	148	3.53	64.5	100.5	100	2.39	43.7
	1.00	.1894	150	141	146	2.46	44.9	100	100	1.69	30.9
Pipe 5.00 feet inside diameter. R=1.25. Area 19.63 square feet	5.00	.9470	148	145	147	5.06	64.2	99	99	3.41	43.3
	4.00	.7576	147.5	143	146	4.49	57.0	98.5	99	3.05	38.7
	3.00	.5682	147	142	144	3.84	48.7	98	98	2.61	33.1
	2.00	.3788	146.5	139	142	3.09	39.2	97	97	2.11	26.8
	1.00	.1894	146	136	141	2.18	27.7	96.5	96	1.48	18.8
Pipe 4.00 feet inside diameter. R=1.00. Area 12.57 square feet	5.00	.9470	143.5	138	141	4.34	35.3	95	93	2.86	23.2
	4.00	.7576	143	137	140	3.85	31.3	94.5	93	2.56	20.8
	3.00	.5682	143	136	139	3.31	26.9	94	92	2.19	17.8
	2.00	.3788	142	133	138	2.69	21.9	93.5	90	1.75	14.2
	1.00	.1894	140.5	129	136	1.87	15.2	92.5	85	1.17	9.6

BASIS OF ESTIMATES.

INTEREST—DEPRECIATION—TAXES AND SINKING FUND.

Interest.

Various issues of New York and Brooklyn long term bonds on or about March 1, 1900, were quoted to net 3.00 per cent. interest; so this rate may be accepted as a proper basis for interest on such bonds as would be required for an extension of the water supply and the same rate may also be taken as the basis of the sinking fund requirement.

Sinking Fund—In the estimate of the cost of the water per million gallons delivered it will be assumed that a part of this cost is the providing each year of a contribution to a sinking fund which shall at the end of forty years be sufficient to pay off all bonds or return all expenditure incurred in the construction of the works. The sinking fund requirement each year to do this, at 3 per cent. interest, is 1.326 per cent.

Taxation—Almost the entire value of the new construction proposed for all the projects of additional water supply now under consideration lies outside the municipal limits of New York.

Taxes upon the property of one municipality used for public purposes and located within another municipality are assessed under different methods in different localities, the most common rule apparently being to tax only on land values, with no appraisal of or levy upon the cost or value of the structures.

The Brooklyn Water Works are taxed in the several Long Island towns in which their collecting reservoirs, aqueducts, wells and pumping stations lie; but only upon the cost or fair value of the land. New York City now pays to other municipalities about $32,000 annually in taxes upon the Croton Aqueducts, storage reservoirs and works connected therewith.

Section 480 of the Greater New York Charter provides that such property shall be assessed and taxed in the counties in which the same is located at the value of the lands, exclusive of the values of the aqueducts and structures, and provides that this assessed value shall not exceed the assessed valuation of other similar lands in the immediate neighborhood.

In reply to inquiry about taxation upon the Hemlock Lake conduit supplying Rochester, New York, the Commissioner of Public Works called my attention to chapter 908 of the Laws of the State of New York, passed 1896, providing for the full taxation of all property owned by a municipality outside of its corporate limits, and stated that a decision has been sustained in

the Court of Appeals to the effect that the city must pay a tax upon its conduit lines and other property, and that taxes are now paid at a rate approximately of $3 per $1,000 of an assessed valuation, which is about half the actual cost.

In Massachusetts, chapter 352 of the Acts of 1893 makes a provision somewhat similar to that which obtains for the Croton and the Brooklyn supplies, and takes only the land, excluding all structures like dams and aqueducts, but fixes the valuation once for all at the average assessed value of the land for the three years next preceding the taking. The amount paid each year while thus reckoned on a constant value is a varying percentage according to the tax rate in the several towns.

In the case of the works under construction by the Metropolitan Water Board, in which a large tract of country is to be submerged and many houses and factories destroyed, similar to the work herein proposed at Dover Plains, a special provision is made for an annual payment to the town in which this property is located, in lieu of taxes assessed in the ordinary manner, thus making recognition of the fact that the town was losing a large amount of taxable property.

While taxation on basis of cost of tunnels and underground masonry would be utterly absurd, there is an element of fairness in liberal compensation to a community for the loss of income caused by reducing the quantity of taxable real estate within the limits of the town or through loss of taxes on the personal property of those who will move to other towns when they sell their homes to the Department of Water Supply. Moreover, the water works, although providing their own sanitary patrol, will share in the general benefits of police protection and the service of the local courts, and its property will receive the benefits of the town's highways and public works probably in larger measure than compensated for merely by a tax on the vacant land. Our estimate should err, if at all, in being too large, and in any such new and comprehensive projects as those now proposed, it will be safe to provide some small margin for special assessments in recognition of the feeling which sometimes obtains in the small town against the large city, and to provide for liberal and generous treatment of this question.

Therefore, in all the estimates on storage reservoirs, an annual allowance of 0.4 per cent. over entire cost will be included to cover taxes and special assessments.

On aqueduct lines there is no such loss to the towns by the removal of taxable personal property and no interference with its business or markets,

and that the several projects may be put on the same basis for comparison, the allowance for taxation and special assessments on aqueduct lines will be made 0.2 per cent. of cost for each aqueduct line as a whole.

It will be noted that in the final summary these percentages give sums that are not unreasonable, and while they are proportionally greater than are now paid by New York City and the Massachusetts cities, are much smaller than that which the City of Rochester has to pay under the law and decision above quoted.

Depreciation.

As a matter of accurate computation of annual cost of the works, or for computing the cost per million gallons of water delivered, it may appear illogical to add depreciation for items which, like underground masonry, do not depreciate noticeably within a half century and which would never require renewal because of wear or decay, but there are general contingencies of abandonment of any one branch of the works, of the possibility of injury through incompetence or mismanagement, together with the remote possibility of injury from earthquakes, cloudbursts or lightning which should not be dismissed without leaving some mark upon annual cost. The Forty-second Street Reservoir and the Old Croton Aqueduct are imperfect illustrations; Boston's expensive Mystic supply, and perhaps Boston's Chestnut Hill Reservoir, are other illustrations; the new danger of electrolysis to water pipes is another; extra work required on Springfield's bad-tasting Ludlow Reservoir after its completion is another illustration that no mistake, in the long run and on the broad view, will be made in providing for even more than can be clearly foreseen.

The current expense and, ultimately, the water rates, as a whole, should be figured to leave as light a burden on posterity as practicable, for even with all these annual charges reckoned against it, a generous supply of good water is wonderfully cheap. And so in all these estimates of cost to the city per million gallons, the yearly expense will be figured with the intention of paying off the bonds and leaving the city in full possession of the works in excellent repair and free of cost at the end of forty years.

Land—On land values, no depreciation will be reckoned, and no depreciation will be figured on cost of extinguishing water rights.

Dams—Masonry dams like those proposed near Merwinsville and Webatuck will be considered permanent. The Essex Company's granite dam across the Merrimack, 35 feet high, 900 feet length of overfall, has in fifty-two years not cost one dollar for repairs, and to-day after the

floods and frosts and summer heat of half a century shows practically no depreciation whatever.

Tunnels—The excavation will be considered permanent. The masonry linings will be considered to require occasional extraordinary repairs and possibly extensions, if, in time, rock over unlined roof shows disintegration. This outlay will be considered equivalent to an annual charge of one-half of one per cent. per year on the masonry. If these renewals happened to be deferred to end of term, the annuity of 0.5 per cent. per year at 3 per cent. interest would amount to the original cost of the masonry at end of about forty-five years, but an accident earlier may come, so the figure of 0.5 per cent. appears safe and conservative. Considering the proportion assumed lined, the cost of lining is found to be 40 per cent. of entire cost of tunnels. We thus figure depreciation on 40 per cent. of the entire estimated cost of the tunnels. At 0.5 per cent. per year on 40 per cent., is equivalent to an allowance of 0.2 per cent. per year on entire estimated cost of tunnels.

For Cut and Cover Aqueduct—The grading is considered permanent and amounts to about 35 per cent. of entire cost of this type of aqueduct. The maintenance charge will keep the earth covering in good form and will repair all petty leaks. The masonry of the aqueduct itself it is assumed may require occasional extraordinary repairs equivalent to an annual charge of 1/3 of 1 per cent. If it happens that these extraordinary repairs could be deferred to end of term, this annuity of 1/3 of 1 per cent. would, at 3 per cent. interest, renew the entire masonry at the end of about seventy-five years. The masonry amounts to 65 per cent. of the entire cost of the aqueduct, so we figure depreciation on 65 per cent. of cost of the cut-and-cover aqueducts at 1/3 of 1 per cent. per year, which is equivalent to an annual charge of 0.20 per cent. on entire cost of the cut-and-cover aqueducts.

Steel Pipe Conduits—As elswhere stated, there are plate iron conduits along the Merrimack river that have been in daily service for fifty years, and which, from my careful inspection, appear good for twenty-five years more, and which were not nearly so well protected against rust as is proposed for these steel pipes, but in a line many miles long, accident from storm or flood, from corrosion, from electrolysis, or injury from causes not now foreseen, may possibly come, and it is well to have the figure safe. Therefore, the life of steel pipes will be called fifty years. On whole value at 3 per cent. interest, the annuity is 0.00887. The occasional repainting of interior of steel pipes is provided for in the estimate for annual maintenance. The extraordinary repairs and depreciation on steel pipes will in round numbers, therefore, be reckoned at 1 per cent. per annum.

Masonry Culverts, Siphon Chambers, Gate Houses and Special Structures—In view of possibility of accident from flood or unforeseen cause, the life will be taken at fifty years. Annuity 0.00887, or for round numbers, call it 1 per cent. per year on entire cost of pipe line and the smaller masonry structures.

Finally, it must be said that the figures adopted for extraordinary repairs, depreciation or renewal on these structures, all of which are of the most permanent character, are properly for insurance against accident and as a safe margin against the unforeseen, rather than allowances for depreciation or renewal in the ordinary sense. We have very little to guide us as precedents in this matter and all the precedents there are, except the Old Croton Aqueduct, which was plainly of a faulty design, whose weakness would not be repeated to-day, tend to smaller allowances than I have assumed. As to the proper magnitude of an allowance looked at as insurance against accident, some experience in weighing the chances of accidents by fire, boiler explosion and otherwise in the far more hazardous conditions of well-built factory property, convinces me that these foregoing allowances, although small, are ample.

LIMITING SIZE FOR AQUEDUCTS.

The question may properly be asked, Should not the large aqueduct lines be constructed in duplicate as a safeguard against accident? or, Should not the water from a source capable of a safe yield of 500 million gallons per day be conveyed to the city by two separate conduits, each of 250 million gallons per day capacity? I have carefully considered these questions from various points of view and it appears to me to be better and cheaper to provide the safeguard in the form of a very large storage of water in great reservoirs like those proposed at Rye and Park Hill, with possibly future additions by raising the Kensico Dam and by perhaps constructing a high dam in the Pocantico Valley as elsewhere suggested; thus obtaining a great reserve of water for emergencies at the *near end* of the main aqueduct large enough so that the conduit can be shut off for two or three weeks. This method works out as cheaper in first cost in the cases that have been assumed, and would tend to economy in maintenance and, as I believe, would afford greater safety. This is the line on which the new Wachusett Aqueduct for Boston has been worked out. With the Old Croton Aqueduct and the New Croton Aqueduct and the Bronx conduit and with a connection also between Manhattan and Brooklyn in future via Park Hill, the safeguards appear ample to admit of building the proposed aqueduct from Sodom to Rye as a single large line.

BASIS OF ESTIMATES OF COST.

Annual Cost of Maintenance of Aqueduct Lines.

This is difficult to estimate with precision, because it may be either doubled or halved by the character of the work undertaken or by the skill and economy of the management. In the following estimate the intention is to be liberal as regards number of men and rate of wages, according to liberal municipal standards, and to treat all the projects on the same basis.

The maintenance of a large steel pipe or of a massive masonry aqueduct buried beneath 4 or 5 feet of earth, or the maintenance of a tunnel 100 feet or more beneath the surface, are very simple propositions. There is little or no work to be done, except to open the gate and let the water run, mow the weeds and bushes over the right-of-way, patrol the line once a day, keep the culverts clean, promptly check wash of embankments, and sweep the interior of the upstream portion twice a year, and the downstream portion once a year or less.

The large annual cost of maintenance of the Old Croton Aqueduct is no criterion whatever toward the cost of maintaining an aqueduct of modern design, since because of its faulty foundation over embankments it was for years continually springing a leak and a comparatively large force of men is continually employed for readiness in emergencies and close inspection and this very large force is kept more or less busy with petty repairs.

But little information is available on cost of maintenance of the New Croton Aqueduct that will serve as a guide in the present inquiry. This New Aqueduct is looked after in connection with the old and is almost wholly in deep tunnel where little care is needed beyond regulating the gate at its entrance. The best guide in actual experience in maintenance of aqueducts similar to those proposed for the extension of the New York supply that now appears available is the recent experience in caring for the several aqueducts that supply Boston.

On the Sudbury, Cochituate and Wachusett Aqueducts of Boston, aggregating nearly 34 miles in length, the regular maintenance force as reorganized in 1898, consists of only about	20 men.
And for sanitary inspections and general care of 9 reservoirs about	20 men.
On office and biological work about	5 men.
At the Terminal Reservoir at Chestnut Hill, in caring for extensive park-like grounds, about	10 men.
A total of about	55 men.

It would appear that for the force on these large aqueduct lines proposed from Pawling to Milltown Bridge, from Sodom to Kensico, and from Kensico to the city limits, a man to the mile, distributed in groups of 4 or 5

at keeper's stations, about 5 miles apart, was a generous allowance for all ordinary purposes along the entire line from Pawling to Milltown Bridge, and from Sodom to Park Hill, a total distance of 12 + 22 + 14 = 48 miles. Add for keepers at Patterson, Milltown, Sodom, Salem, Cross river, Byram and Greenville, seven in all. Add a team and driver extra at each of these keeper's stations. Add for extra help in sweeping the aqueduct twice a year (aided in this by a machine) and allowing liberally for tools and supplies, I estimate that $1,100 per mile should provide liberally for all necessary care along the aqueduct lines. Add to this liberally for contingencies and adjusting the figure a little in proportion to the size of the aqueduct—wider embankment to be trimmed, larger interior to be swept, more numerous siphons to be cleaned and repainted at intervals of perhaps ten years, and the wider culverts to have face joints repointed at rare intervals—we will, for all aqueducts cared for in the best manner, with slopes neatly trimmed and proper cleaning of interior, assume as a safe outside figure for the annual cost of maintenance of all aqueducts, including tunnels:

For	250	million	gallons	maximum	capacity....	$1,250	per	mile	per	year.
	500	"	"	"	"	1,500	"	"	"	"
	1,000	"	"	"	"	1,750	"	"	"	"

CARE OF RESERVOIRS.

On the Ten Mile impounding reservoir, exclusive of the Housatonic, for purpose of an ample preliminary estimate we will assume:

General Superintendent.....	1 Hydraulic Engineer.
At outlet sluices.....	1 Gatekeeper.
Gatemen, working 8-hour shifts.....	3 Gatemen.
At River Dam, working 8-hour shifts.....	3 Watchmen.
Sanitary patrol, Ten Mile Division.....	7 Inspectors.
Laborers and foremen, on care of margins...	6 Laborers.
Engineer party on water gaugings and miscellaneous surveys.....	4—1 Engineer and 3 Assistants.
Biological party on water analysis, etc.....	3—Biologist, Chemist and Assistant.
Teams for sanitary patrol / Team for engineer party	1 Hostler.
Steam launch for sanitary patrol in summer..	1 Boatman.
Say, on a very liberal basis, in all.....	30 Men.

At Rye and Park Hill, equalizing and distributing reservoirs.....	2 Keepers.		
	2 Sanitary Inspectors.		
	6 Gatemen.		
	5 Laborers.		
Total	15 Men.		
A liberal estimate for salaries, wages, tools and supplies of every description for these 45 men and the necessary teams, appears to be.....		$60,000	per year.
On Ten Mile project, including delivery of the East Branch water at high pressure, we thus have as the total maintenance charge per year, 48 miles of aqueduct maintenance, at $1,250.....		60,000	" "
Maintenance and sanitary inspection of all reservoirs as above		60,000	" "
Total		$120,000	" "
With the Housatonic added, the expenses on the Upper Basin might be so increased, with the greater work of sanitary inspection, as to bring total cost to.....		$80,000	" "
Maintenance of 48 miles of aqueduct.....		72,000	" "
With 500 million gallon aqueduct a total of.....		$152,000	" "
Or, with 1,000 million gallon aqueduct, a total of, say.		164,000	" "

Appendix No. 6.

TEN MILE RIVER SUPPLY—QUANTITY AND COST.

The Ten Mile River, with a dam three-quarters of a mile above Webatuck, as shown by computations on page 347, can be relied upon to supply 150 million gallons per day throughout a series of years of extremely low rainfall, and all of this additional supply will flow by opening a sluicegate at Pawling, into the present East Branch Reservoir of the Croton Watershed, and so on down into the Croton Lake, a part to be carried thence to New York for the immediate future in the present aqueducts, which together, when clean, have a surplus capacity of 105 million gallons per day over and above the safe permanent yield of the Croton Watershed. No other source of supply found on the map equals this in directness, cheapness, simplicity, economy of operation or facilities for quick construction.

This Ten Mile Watershed has, by the concurrent testimony of many connected at one time or another with the development of New York's water supply, long been looked upon as the most available extension next in order for the supply of Manhattan and Bronx, and after reviewing its possibilities of abundant storage development, its impervious dam site, its proximity to present watershed and the fact that by merely building one dam its waters can be turned into the present aqueducts, I believe it certain that for a supply of only 150 million gallons per 24 hours, this is the best and cheapest extension possible. Indeed, so obvious are its advantages that little beyond an inspection of the map and plain common sense, apart from elaborate estimates, is needed to secure its favorable consideration. The obstacles are mainly of a legal nature, and arise:

First, from the diversion of water which now finds its way into the State of Connecticut, just as the Nashua river now being developed for Boston's supply finds its way into New Hampshire, and just as the water from Kettle Brook recently taken for the supply of Worcester formerly flowed into the State of Rhode Island, furnishing water to many valuable mills. The legal difficulties were all readily solved in the case of Boston and Worcester; it is reasonable to expect that the similar difficulties in the way of New York and the Ten Mile could be solved.

The second legal difficulty is that the Ramapo Company has filed a dam and reservoir location a few miles downstream from Dover Plains, which is not an advantageous one for the best development of the watershed for New

York's supply, but which would be in the midst of the reservoir site herein proposed.

The third obstacle is the unfortunate necessity of disturbing many home-sites, very much as had to be done for the Massachusetts Metropolitan Supply; for which several villages had to be flooded. Many large works of public improvement are subject to the same unfortunate necessity, and in the Croton Watershed many houses have had to be taken for the public welfare to prevent contamination of the water supply. It is impossible to find any sufficient source, except the Hudson river and the ground waters of Suffolk County, Long Island, within a hundred miles of New York, whether Fishkill, Wallkill, Ramapo or Esopus, where similar disturbance of homes will not have to be accomplished. To build the Esopus dams high enough to give it the same yield of 150 million gallons daily, would destroy a very large number of houses. The best remedy for this is to deal generously and give each owner enough to build himself a new and better house and something besides. The city can well afford to do this to secure so favorable a location.

Previous Plans of Development.

The plans for developing the Ten Mile river watershed heretofore proposed, notably those set forth in 1896 in the Report of Investigations for Extension of Water Supply of the City of Brooklyn, which only looked for 100 million gallons per day, do not appear to be the best that can be devised for its full development to supply 150 million gallons per day.

It now appears that the best plan for making the whole of this watershed available is by a single high dam located about ¾ mile westerly from the little village of Webatuck, N. Y., about 1½ miles upstream from the confluence of the Ten Mile river with the Housatonic, thus raising the water surface to an elevation of 450 feet above sea level when the reservoir is full, and so planning the outlet channel past Pawling that this reservoir can in time of severe drought be drawn down to elevation 425. or 420. above sea.

Although I have not had opportunity to make borings in the river bed at the dam site, the exposure of ledge on both the adjacent hillsides and an exposure of ledge bottom in the river half a mile upstream and three-quarters of a mile downstream indicates that this dam could be constructed of masonry resting on a sound ledge. This Webatuck dam would not be very much larger than the Titicus dam in the Croton Watershed.

THE PLAN OF DEVELOPMENT OF THE TEN MILE WATERSHED FORMERLY PROPOSED FOR THE SUPPLY OF BROOKLYN, N. Y.

Referring again to the report on water to be obtained from the Ten Mile river, set forth in Report on Future Extensions of the Water Supply of Brooklyn, transmitted to the Common Council January 31, 1896, by Alfred T. White, Commissioner of City Works, and printed in a volume of about 100 pages, 12 inches by 14 inches:

On pages 16, 17 and 42 of the said report is given a brief description of the method of development proposed, and in Table No. 1, page 46, are given the areas of watersheds, the reservoir elevation and capacity, and on page 47, Table No. 3, is presented the estimate of cost of a supply of 100 million gallons per day. Of this estimate of cost of supplying 100 million gallons daily (or perhaps 127 million) of Ten Mile water for the supply of Brooklyn, only the two following items will apply to the supply of water from the same source to Manhattan and The Bronx, namely:

" Estimated cost of storage..... (Understood to include lands and dwellings flooded, costs of structures and all expenses for development of reservoirs of 19,235 million gallons available capacity and 35,097 millions total contents.)	$4,000,000 "
" Injury to water-powers.....	200,000 "
Total for water rights and reservoirs complete.....	$4,200,000

This includes no allowance for the outlet conduit.

For the more extensive and much more complete works herein proposed my estimate of probable cost is more than double the figure just quoted. But I believe its proposed structures more certain to be tight, cheaper to care for, simpler to regulate, and, on the whole, more economical than the series of reservoir dams planned in the report just mentioned. A part of the difference is due to a more liberal basis of estimating damages and to now providing for stripping a large portion of the reservoir bottom to improve the quality of the water, and in part apparently to some unreliable data used in the earlier figures.

No details are given in the report as to how these sums in the Brooklyn estimate were reached, excepting the statement on page 16: " In estimating the cost of reservoirs we have assumed $200 as the cost per million gallons, which was Mr. Allan Campbell's estimate of the cost of those on the Croton Watershed." Obviously, cost per million gallons in one reservoir is no safe basis upon which to estimate cost of dams, lands and legal expenses, for another reservoir, for with the same number of gallons capacity in each of two reservoirs the dam for one may cost four-fold what the dam will cost for

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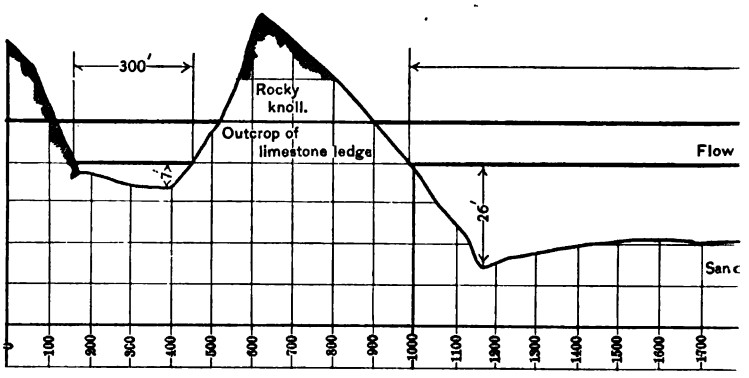

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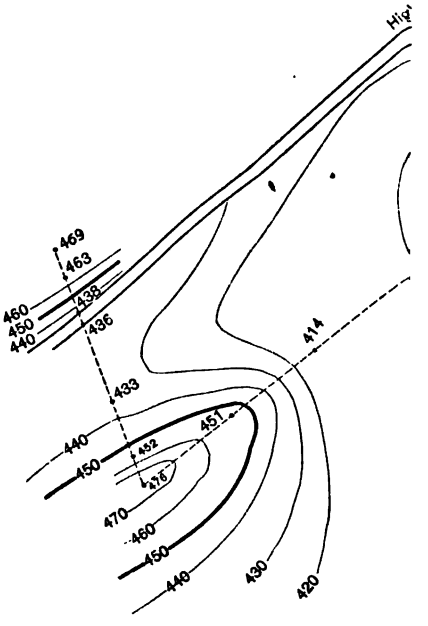


the other, according as the dam site is favorable or unfavorable, and the valley above it wide and long, or short, steep and narrow.

I began my investigations of the feasibility of the Ten Mile River supply along the same lines of development proposed in the Brooklyn report, but on taking the plans presented in Plates No. 6 and No. 7, and described in pages 16, 17 and 46 of this Brooklyn Report on to the ground, I was at a loss to understand how certain of these dam sites could have been chosen, particularly the dam site one mile north of Dover Plains, and after studying into the matter, was led to question its estimates of cost, and to think them the results of a more hasty reconnaissance than at first sight appears, and was led to believe that the assistant who made that reconnaissance made some mistakes in his survey. The ground speaks for itself to any engineer who will visit it. A line of levels run over it by Mr. George A. Taber, C. E., gives the section on the following page.

A new examination was, therefore, begun in the effort to find the best possible site for the *full* development of this watershed, taking a broad view also toward the future, recognizing that even the whole of this would suffice for less than ten years' growth for Manhattan and Bronx alone.

Examination was begun by exploration for reservoir sites on the tributaries further away from Dover Plains. Apparently good sites exist which could be developed piecemeal and utilize perhaps half or two-thirds the capacity of the stream. This study was not completed, because brief study showed that the best site for the requisite storage to develop a fair proportion of the whole flow must be found somewhere on the main stream, or lead to the same losses found on the Croton prior to completion of the New Croton Dam, where, for example, on February 5, 1900, and again on February 13, 1900, water ran to waste at the rate of 3 billion and 2 billion gallons per day, while the reservoirs on the tributaries were not yet half refilled. The same volume of storage may not be worth half as much if located on the head waters of a tributary as if located at the outlet from the watershed.

The following possible dam site sections were examined and roughly surveyed.

Near the State line three possible sites, as follows:

Site A. Almost exactly on the State line. would require roughly 400,000 cubic yards of masonry, and cost, roughly, say $2,600,000.

Site B. About 600 feet above State line—looks more attractive on casual inspection, but widens out so much at top as to require more masonry, and outlook for ledge bottom less favorable than at A. Requires also a back dam to close high valley.

Site C. About 1,000 feet downstream from State line. Good outcrop of ledge, but apparently less economical than A.

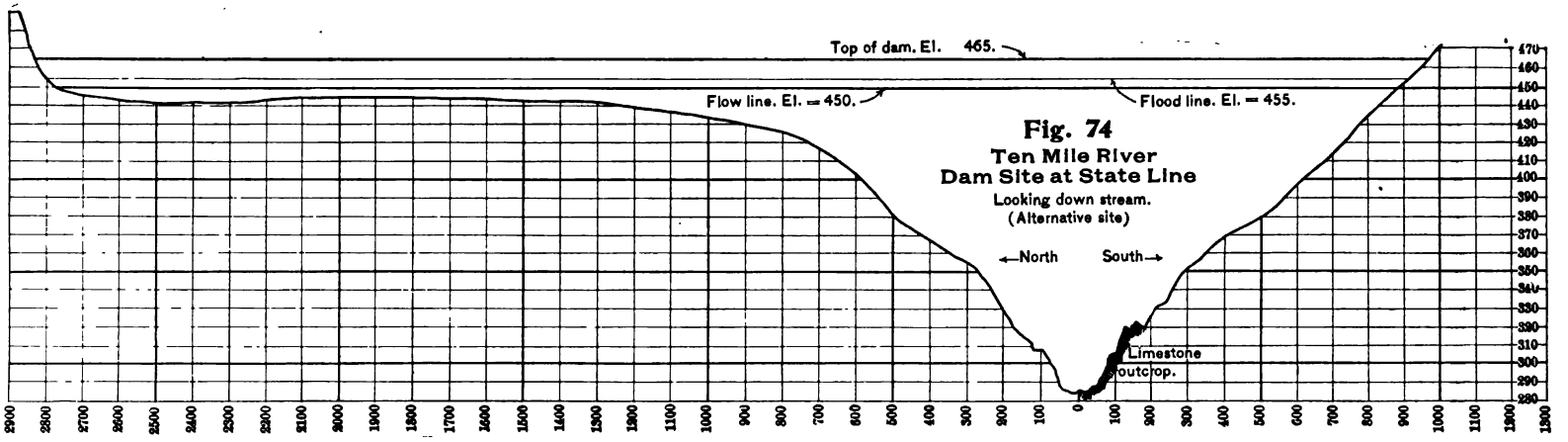


Fig. 74
Ten Mile River
Dam Site at State Line
Looking down stream.
(Alternative site)

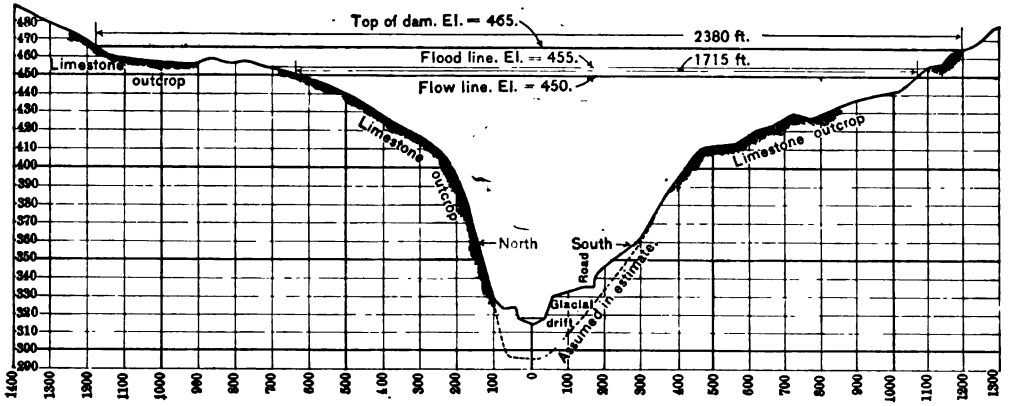


Fig. 75
Ten Mile River Dam Site ¾ mile above Webatuck.
Looking down stream. (Estimates based on this location)

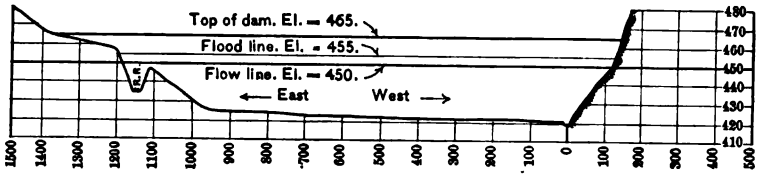


Fig. 76

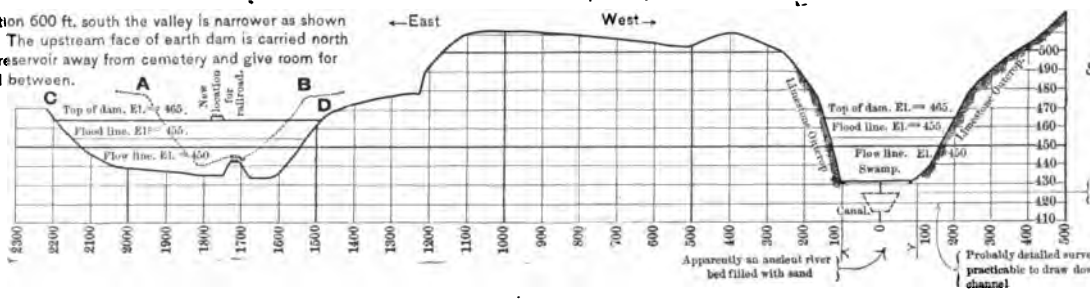
Alternative Site for Outlet Dam at Hurd's Corner.

Fig. 77

Site for Outlet Dam just above Pawling Village

This site adopted in plans and estimates.

On section 600 ft. south the valley is narrower as shown at A-A. The upstream face of earth dam is carried north to get reservoir away from cemetery and give room for railroad between.



Limit of draft in greatest drought for either Housatonic or Ten Mile.

Probably detailed surveys with test pits and borings will show it practicable to draw down to 420 without undue expense for digging channel

SECTIONS of DAM SITES for TEN MILE RESERVOIR

Surveyed for J. R. Freeman, C.E. by Geo. A. Taber, C.E. Nov. 1899.

Following upstream, no particularly good sites are found until ¾ mile upstream from Webatuck, where the excellent site exists which was finally adopted as the basis of these estimates (and which I subsequently learn had also been noted as a favorable dam site by Mr. Birdsall). No site further upstream and below the confluence of the Swamp river and the Ten Mile was found that appeared nearly so economical as this for a high dam flowing the whole river back through Pawling into the Croton. The drainage here at Webatuck is 7 per cent. smaller than at the State line site, but the best data now at hand indicate that a dam at Webatuck will cost about one million dollars less than a dam at the State line site, because of the smaller volume of masonry required to close the gap.

From such surveys as we found time to make it appears certain that the dam site ¾ mile above Webatuck is the best that the Ten Mile river affords. Taking it as the basis of our estimate of cost is certainly safe, because if a better site or series of sites is found, they will be judged as better because of a reduced cost. While a more detailed exploration by instrumental survey would naturally be made before becoming definitely committed to the Webatuck location, it now seems plain that this location presents sufficient adavantage over any possible series of small reservoirs and canal around Dover Plains to more than justify the expense of buying out these houses and changing the railroad location.

ESTIMATES OF YIELD OF TEN MILE RIVER WATERSHED.

With dam about 150 feet high located about three-quarters of a mile above Webatuck. Areas of watersheds and reservoirs taken by planimeter from maps of U. S. Geological Survey.

Total area of watershed.....	200.4 square miles.
Area of water surface in proposed reservoir with flow line at elevation 450 feet above sea...	15.0 square miles.
Probable additional evaporating surface in drainage area outside of reservoir limits, consisting of ponds, streams and one-half the swamp area (roughly).....	2.0 square miles.
Average area of reservoir as partially emptied under conditions of complete use.....	13.0 square miles.
Total average extent of water surface in watershed with reservoir at probable levels required in maximum use.....	15.0 square miles.

Net land surface in watershed.....	185.0 square miles.
Per cent. of water surface to land—average conditions	8.1 per cent.
Storage volume between elevation 450; proposed flow line and elevation 425 (low water level in maximum drought).....	61 billion gallons.
Storage volume per square mile of net land surface	*330 million gallons.
Rate of uniform draft per square mile of net land surface finally adopted as best adapted to utilize resources of this reservoir to fullest extent consistent with preserving quality of water without impairment from weeds growing on margins when reservoir is too frequently kept below flow line for long periods:	
Per square mile of gross area.....	750,000 gallons per day.
Or per square mile of net land.....	810,000 gallons per day.
Safe yield per day from this entire watershed of 185 square miles net land corresponding to above figure of 810,000 gallons per square mile—185 x 810 = ...	**150 million gallons.**

This 150 million gallons per day is the figure finally adopted by J. R. Freeman for the safe yield in the most severe drought, and cycle of years of smallest rainfall, of this Ten Mile River Watershed with dam three-quarters of a mile above Webatuck, and reservoir having area and available depth as given above.

In proportion to gross area of watershed this corresponds to a safe yield of 150 x $\frac{180}{100}$ = 270 million gallons per day from the Croton Watershed after the completion of the new Cornell Dam.

Probable greatest length of time in course of half a century that the reservoir level would ever be continuously below spillway level with this rate of draft, about.....	3 years.

The Sudbury records indicate three or four years—the Croton records for thirty-two years indicate two or three years as the longest period of depletion, with this rate of draft of 750,000 gallons per square mile gross area, and this per cent. of water surface. See Table No. 31, page 232, also Diagrams Nos. 46 and 47.

* In these estimates of safe yield of the Ten Mile and Housatonic rivers I have closely followed the Croton estimate of safe yield of 275 million gallons per day from 360 square miles, but for confirmation have followed the method of estimate of F. P. Stearns, C. E., as formulated and explained in the Report of the Massachusetts State Board of Health, 1890, page 348 *et seq.*

This Stearns method, by the aid of diagrams like Fig. 48 and Fig. 49, is very easily applied and is a much more logical method and more accurate than the old way of assuming a certain number of inches of rainfall as collectible, and available with the given storage capacity; and then distributing this through an assumed number of dry months.

Check Estimates for Comparison.

SUDBURY BASIS.

Maximum uniform rate of draft per square mile of net land possible with this storage volume per square mile and this percentage of water surface exposed to evaporation according to Sudbury records for past twenty-five years, see Diagram No. 49	880,000 gallons.
Greatest length of time that reservoir level would have ever been continuously below spillway in past twenty-five years at this rate of draft, per Sudbury records (January, 1879, to December, 1885), draft 815,000 gal. sq. m. gross area	7 years.
Safe yield from this entire watershed on Sudbury basis 185 sq. m. x 880,000 gal. =	163 million gallons per day.

CROTON BASIS.

Maximum yield as confirmed by Diagram 48, from Croton record—	
Total available storage per square mile of gross area	305 million gallons.
Ratio of water surface to gross watershed area	7.5 per cent.
Safe yield per square mile by Croton record "as occurred," Diagram 48	920,000 gallons.
Safe yield entire watershed, 920,000 x 200=	184 million gallons per day.
Safe yield square mile by Croton record "as adjusted," Diagram 48	855,000 gallons.
Safe yield entire watershed, 855,000 x 200= —according to the adjusted Croton records of Fig. 48.	171 million gallons per day.
Period of depletion per Croton record "as occurred"	6 years.

From the above check-estimates it appears that with this very large volume of available storage, it would be safe to rely upon a daily draft of about 170 million gallons per day instead of the 150 million gallons per day which I have adopted as a conservative figure.

The reasons in favor of the smaller figure are similar to those which lead me to call the safe Croton yield 275 instead of 300 or more million gallons per day; mainly the fear of impairment of quality of water by keeping a reservoir too long drawn below high-water mark.

Whenever this objection is removed by filtration, then a larger figure may be adopted for the safe yield.

PROBABLE COST OF A SUPPLY OF 150 GALLONS PER DAY FROM THE TEN MILE RIVER, WITH DAM ABOUT THREE-QUARTERS OF A MILE ABOVE WEBATUCK (OR ONE AND ONE-HALF MILES UPSTREAM FROM BOUNDARY LINE BETWEEN NEW YORK AND CONNECTICUT).

(The details of the following rough estimates are stated with more than ordinary fullness in order that whatever margin of uncertainty there is may be clearly apparent, and that it may be the easier to critically examine, check or modify them. It is to be kept in mind that they are based merely upon a careful reconnaissance in advance of detailed surveys.)

Cost of Masonry Dam (See Fig. 75).

Dimensions above present surface of ground are based on surveys and levels in October, 1899, by George A. Taber, C. E., under my supervision. The line of levels upon which these elevations and estimates are based were started from bench-marks kindly given us by the New York Water Supply Department, and were checked across country by Mr. Taber to the bench-

marks and levels of New York, New Haven and Hartford Railroad near Merwinsville.

Form of dam proposed is similar to upper portion of Housatonic straight dam (Fig. 80). For cross-section of valley see Fig. 75.

There is an excellent outcrop of hard limestone ledge on both sides of the Ten Mile river at this proposed dam site. No borings were made to ledge in bed of river, but excellent indications that it is not deep are found in the bare ledge bottom of the river half a mile above dam site, and again found three-quarters of a mile below this site, and in the level limestone formation. The probable excavation below the present surface was estimated at 20 feet near middle, according to judgment made while on the ground in company with an expert geologist, from the visible outcrops of ledge and from the apparent geological formation (see Appendix No. 16, p. 587).

Probable total masonry in Webatuck dam.....	214,000 cubic yards.
The unit cost, $7.50 per cubic yard, adopted in this preliminary estimate for Webatuck Dam, complete, in view of the possible distance of a good granite quarry, is believed to be liberal.	
Estimated cost of dam located ¾ mile above Webatuck 214,000 cubic yards, at $7.50 per cubic yard of entire bulk.....	$1,605,000.
Titicus Dam, built by Croton Aqueduct Commissioners in 1890–4 contained 208,450 cubic yards, cost.....	$933,065.
Total cost per cubic yard of Titicus Dam, including earthwork, masonry, gate-house fittings and clearing up.....	$4.47 per cubic yard.
New Croton Dam, if containing 614.000 cubic yards, as originally expected, at contract prices would have cost.....	$6.76 per cubic yard.

LAND DAMAGES AND COST, TEN MILE DIVISION ONLY.

Areas Required to be Taken for Reservoir Purposes with Dam on Ten Mile River Three-quarters of a Mile above Webatuck and Flow Line 450 Feet above Sea Level.

Basis of estimate—cultivated land, $100 per acre, including legal and other expenses. Pasture land, $30 per acre, including legal and other expenses. This from various examples of actual sale appears 50 per cent. above fair market value of average. Some lots worth more, some less.

The following amounts of land required include large marginal areas

to be taken for protection of the quality of the water, around the entire circumference of reservoir, estimated to 250 feet outside the flow line on steep hillsides, and to 500 feet outside the flow line on flat slopes, and including nearly all of the peninsulas.

Values of buildings and dwellings were estimated by hasty external inspection of each by Morris Knowles, C. E., and adding 50 per cent. to the probable fair market value.

TABLE NO. 50.

	LAND.	BUILDINGS.
Town of Amenia—		
Land—5.2 square miles, 3,330 acres; total for land.....	$234,000	.....
159 dwellings and other buildings; total.....	.....	$369,000
Town of Dover—		
(Including about 130 acres land and one dwelling in Town of Washington.)		
Land—17.1 square miles, 10,950 acres.....	911,000	.....
336 dwellings and other buildings.....	.....	689,000
Town of Pawling—		
Land—1.6 square miles, 1,024 acres.....	71,000	.....
18 dwellings and other buildings.....	.....	33,000
Total for 513 dwellings.....	$1,216,000	
15,300 acres of land.....	.....	$1,091,000
(Many of these farm buildings are small, old and out of repair, and the fair value of these lessens the average cost.)		1,216,000
Total for land and dwellings and accessory buildings, within and on the margins of the proposed Ten Mile Reservoir.....		$2,307,000

From such few of the buildings and such of the lands in the Ten Mile basin as I inspected personally with a view to valuation, I think it probable these prices are as high as those at which lands and buildings were acquired by the Massachusetts Metropolitan Water Board; but this item more than any other in the estimate is subject to errors of judgment and to being increased by unnecessary friction or extravagant methods in settlement, and should be reviewed by local experts.

About 360 buildings, including dwellings and factories, were taken for the Massachusetts Metropolitan Works for the Wachusett Reservoir to supply 111 million gallons per day to Boston and vicinity. Out of all the dwellings and lands acquired by the Massachusetts Metropolitan Water Board, three-fourths were bought and paid for under an amicable agreement as to price—*without recourse to litigation, eminent domain or referees*, and with no noteworthy expense for experts outside the office force; whence it may be assumed that the prices paid were reasonably liberal.

Special Damages on the Ten Mile System, Mill Sites, etc.

For obvious reasons the estimates on the following items (which were rough) are not presented separately:

(*a*) Add for three abandoned mill sites and four mills and mill sites in use within basin.

(*b*) Also include injury to quarry buildings and unused quarry site (the South Dover Company's quarry would not be impaired by this Ten Mile-Webatuck plan, but their Stone Polishing Works would have to be moved, to conform to change of railroad location, and their trolley road would have to be extended).

(*c*) Also including possible claims for injury to certain small water-powers and water-rights on the Ten Mile river and its tributaries.

(*d*) Also for possible claims for impairment of established business.

(*e*) Also include general contingencies of possible underestimate of damage on lands, buildings and on impairment of business.

The above items, *a* to *e*, inclusive, at a rough estimate aggregate..	$400,000
Possible awards of damages for inpairment of water-power on the Housatonic river by diversion of Ten Mile river only, including legal and expert expenses on same, for 3 developed powers and 5 proposed developments, say.....	500,000
For moving 5 (or possibly 6) cemeteries, public, semi-public and private, to new site, landscape development and reinterment, and cost of land for new site.....	80,000

From counting the number of headstones there appears to be in all about 1,800 interments (the number moved on the Metropolitan work at Clinton, Mass., is about 4,500). Experience in other places indicates this $80,000 is a liberal estimate for doing this work in a careful and appropriate manner, with due respect and observance of all the proprieties. The above count includes a count of the 600 in South Dover Cemetery, all of which, being 50 feet above the flow line, would not necessarily be disturbed. Since a sentiment for its removal might develop, it was included for purpose of obtaining an outside figure.

Removal of bushes, trees and stumps from entire flooded area (exclusive of swamps and south arm provided for in estimate of cost of stripping), aggregate area of wood and brush land, about 860 acres at $70..... $60,000

(A contract for grubbing 850 acres in Massachusetts Metropolitan Wachusett Reservoir was awarded June 13, 1899, at $70 per acre.)

Removal of organic matter from sites of 513 dwellings and farmyards of portion not completely stripped, less value of old material. Removal of mud and muck from beds of 3 mill ponds of about 50 acres total acrea, or covering deepest deposits with clean sand..... 75,000

Stripping loam and muck and vegetable matter from southwest arm of reservoir for about 10 miles in length upstream from outlet at Pawling for its entire average width and filling shallow flowage, about 4.9 square miles, 3,100 acres at $700 per acre 2,170,000

(There is much uncertainty in this item because of interruption of a reconnaissance or rough survey of this territory, and which in pressure of other matters was not resumed.)

According to past practice on New York City's reservoirs this item could be omitted, but, in my opinion, this stripping of reservoir bottom with its probable prevention of growth of algæ, etc., and the bad taste arising from their decay, which does now occur occasionally in some of New York's reservoirs, would be wise, and should be included in any preliminary estimate to this extent at least.

(This 3,000 acres is only 73 per cent. of area now being stripped in the Wachusett Reservoir of the Boston Metropolitan Supply at an estimated cost of $2,909,600.) Contracts have been recently awarded at 26 cents to 35 cents per cubic yard for stripping 2,013 acres. On the estimated average depth of about 14 inches, these Boston contracts aggregate 3,900,000 cubic yards and average about $610 per acre.

Change in location of railroad. Beginning ¼ mile south of Pawling Depot, raise tracks on their present location ; retain present location of depot; follow present track location northerly to a point just beyond cemetery ; thence follow reservoir margin to a point near Hurd's— cross south arm of reservoir at Hurd's ; thence follow up westerly margin of reservoir on a new embankment made in part from material taken from channel-cut and in part from the stripping of reservoir bed, about 16.5 miles of main line, plus 1.5 miles sidings, including the whole work involving land, heavy side hill cuts, embankments, bridges, tracks, station buildings, telegraph and fences.

The following estimate is based on a brief reconnaissance by Horace Ropes, C. E.:

Grading 16.5 miles new location and 0.5 mile raised on old location, including riprap where needed. Fills on reservoir margin made in part from reservoir strippings faced with clean gravel and riprapped where needed, half or more of distance being in heavy side-hill work (17 miles at average $35,000 per mile)	$595,000	
1.5 miles sidings, grading	15,000	
Bridges and trestles and culverts, one bridge crossing narrow arm of reservoir at Hurd's, 3 highway crossings, six 10-ft. arch culverts, 35 2 x 2 culverts	150,000	
Track—16.5 miles main track on new location, laid and ballasted; 0.5 mile raised and relaid in present location—at $8,000	136,000	
1.5 miles sidings, at $6,000	9,000	
2 new station buildings (combination freight and passenger) and raising Pawling Depot	12,000	
Fencing—equivalent to 20 miles single fence (woven wire), at $200	4,000	
Land not covered in reservoir location	5,000	
	$924,000	
For an outside figure call it equivalent to 17 miles at about $59,000 per mile		$1,000,000

(Possibly it might be desired to shift the line from 1 to 2 miles further west, or west of the range of hills bordering the reservoir if local traffic was considered more desirable than a low-grade line for freight from Chatham.)

If so, the above estimate is believed sufficient to cover the cost, a brief reconnaissance having been made over the more difficult portion of such a line.

New highways on both sides of Ten Mile basin—Compensation estimated equivalent to about 35 miles of highway, at $20,000 per mile..... $700,000

As a matter of providing a strict equivalent this figure is perhaps $500,000 too large. The 35 miles of road discontinued are of a quality that could probably be built for $2,000 per mile, and a road of same character that the Croton Aqueduct Commissioners are understood to have built in Croton basin, could probably be built for $5,000 per mile. The broad macadam roads built around certain of Boston's reservoirs have cost fully $20,000 per mile, and the actual cost may range anywhere between these two figures according to the kind of road built. The broad macadam road, with smooth easy grades, is an extravagance for this region, unless viewed as compensation or liberality toward the town in return for submerging so much territory. The method of compensation by building good macadam roads in such locations as desired by the town, near to the reservoir, appears as good a way as any to add to the attractiveness of these towns and aid their growing industry of keeping summer boarders, and in full reparation for any injury.

Earth-fill causeway across basin at Wing, filled in part from strippings, slopes to be graveled. Also earth-fill causeway south of Wassaic.

(Profiles of basin at these causeways at Wing and Wassaic not surveyed. Depth estimated from contours of geological survey maps corrected by known elevation at centre of valley at Wing. The section at Hurd's was surveyed while it was being considered as an alternative dam site.)

Also Highway Bridge at Hurd's—total for 3 crossings..... $175,000

Outlet Dam at Pawling.

It is proposed to locate the outlet dam on a line about 800 feet northerly from the Pawling Railroad Depot, at the central point shown in the photograph, Fig. 78.

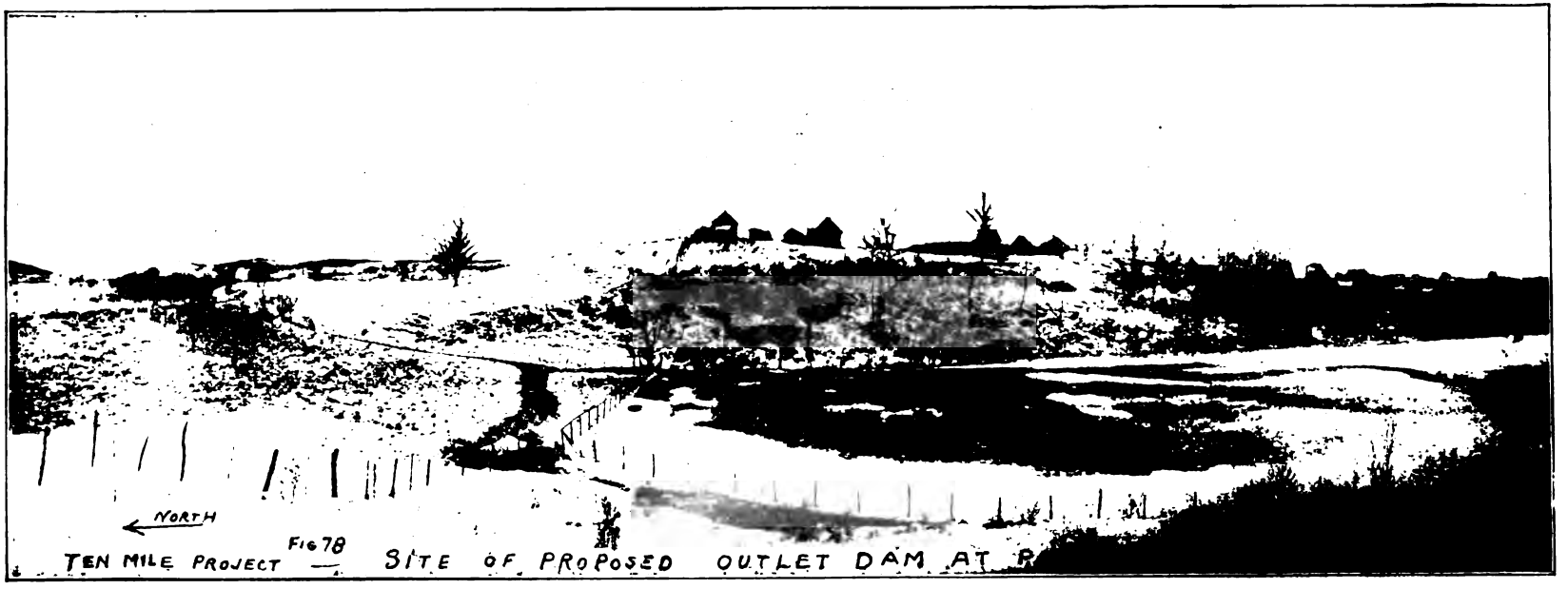
Estimate is based on surveyed section across valley without test pits or boring. Ledge outcrops in excellent form at both ends of proposed main outlet dam across this old channel; this outlet dam, location about 1,200 feet west from the railroad. A second gap through which the railroad now runs requires closing by an earth embankment with a core wall. See Fig. 77.

Both these Pawling dams are included in the figure below. Both are very simple propositions requiring no spillway. Location appears remarkably favorable.

Sound ledge or suitable base for core wall assumed 30 feet below surface in centre for 300 feet, and assumed 20 feet below surface for 300 feet.

Outlet dams and dike to be of earth, with concrete core wall; greatest height of main outlet dam above present surface about 35 feet. Total length of both dam and dike is about 700 feet. This figure includes also a broad fill up along the margin of cemetery made with earth from the channel excavation—total 85,000

Masonry sluiceways, gates, hoisting apparatus and gate-house; masonry portion sufficient for ultimate addition of Upper Housatonic; also office and residence for keeper—total.... 50,000



TEN MILE PROJECT — Fig 78 — SITE OF PROPOSED OUTLET DAM AT R

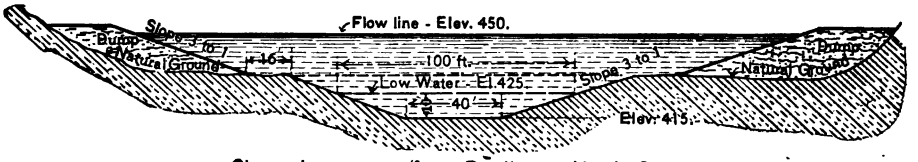
Channel Upstream from Outlet Dam at Pawling.

For drawing reservoir to Grade 425.

The following estimate is based largely upon the United States Geological Survey Maps, which were checked by a stadia reconnaissance by G. A. Taber, C. E., over a part of the line. A survey was started and run for about a mile up over this line, when interrupted by bad weather; there has been no opportunity yet found to complete it, but the following estimate is believed liberal. The elevation of ground along Croton East Branch and Swamp rivers was also determined in part from line of bench marks and elevations kindly furnished through courtesy of Mr. Birdsall by Col. Meehan, Topographical Engineer of the New York Department of Water Supply.

A broad lawn is proposed to be filled in between the Pawling Cemetery and the new location for the railroad and the margin of the reservoir from material dredged from the canal above the sluices, and the eastern end of the East Dike is swung to the northward for that purpose.

Deep channel upstream from Pawling into main reservoir above Hurd's, as per section above, bottom at elevation 415;



Channel up-stream from Pawling to Hurds Corner. **Fig. 84**

for drawing Reservoir down to Elev. 425. Size sufficient for Ten Mile plus Housatonic.

bottom width 40 feet—of size and depth suitable for addition of Upper Housatonic river in future, as per Fig. 84.

Earth excavation on basis of above sketch, not far from 400,000 cubic yards, at 25 cents.....	$100,000
Assume ledge, 5 per cent., 20,000 cubic yards, at $2..	40,000
	$140,000
Or, to be safe, and to allow for depositing a portion of excavated material in embankment between Pawling Cemetery and Reservoir, say.....	$200,000

(It is quite possible that detailed surveys and test pits will show that by modern methods of excavation by steam shovels or dredging this channel can be excavated for one-half of this sum.)

Open Canal from Pawling Outlet Dam South to East Branch Reservoir.

The natural features are remarkably well adapted for conveying the Ten Mile or Housatonic water from the outlet dam, southerly into the present East Branch Reservoirs. As shown in the photograph, Figure 78, there is a clearly defined ancient river bed, now a swamp, running southerly all the way from the outlet dam down to a point about half a mile or more below Pawling, where it gradually broadens into a wet, nearly level swamp. The topography and certain geological features strongly point to a river much larger in volume than the present Ten Mile at Webatuck, having thousands of years ago flowed southerly down past Wing, Pawling and Patterson, into the East Branch of Croton river, along the line now proposed for this canal. The report of the geologist, Professor Crosby, on this subject, found in Appendix No. 16, page 583, is very interesting. Indications are strong that the excavation for this canal would all the way be in soft, wet sand, that could be excavated by a floating dredge of the centrifugal type or endless chain and bucket type, starting at either end and at a cost per cubic yard not more than half that at which this work is figured in the estimate below.

I made personally a brief reconnaissance over a portion of this line, and a line of levels and measurement was begun by Mr. Taber down along this proposed canal, over the frozen surface of the swamp, in order to more closely estimate the cost of excavation, but this stadia survey was interrupted by the sudden flood of February 5. The swamps were again flooded February 13 and remained impassable for survey for some weeks.

A reconnaissance was subsequently made by Horace Ropes, C. E., who tramped over or near to the proposed canal line for its entire length from Pawling to a short distance below Deforest Corners, armed with a steel sounding rod and probed the depth of the mud and examined the characteristics affecting cost of excavation. Substantially the whole appears to be material that could be handled by a dredge and deposited as in Fig. 85 at a cost of say 10 cents per cubic yard.

Mr. Ropes' report of this exploration is as follows:

I examined the East Branch valley from Pawling south for 10 miles.

Through the Village of Pawling, for about ½ mile, the bottom of valley is only from 175 to 250 feet wide; rock shows at points in the bluffs on each side. In this first ½ mile south of the proposed outlet dam, 4 soundings showed from 3 to 6 feet of sand, overlying what felt like hard packed gravel.

From the crossing of the road running south from Pawling to the end of mile 2 from the outlet dam, the valley grows wider. The bottom, varying from 600 to 1,500 feet wide, becomes swampy in character and seems to be a pocket of deep black muck. My 12-foot sounding rod failed to touch bottom at points which were generally nearer the edge than the centre of this swamp. Mr. J. B. Dutcher told me that when he reclaimed a small portion of this swamp, near its head, he drove a scantling 24 feet long down its full length without reaching hard bottom. For the next 1½ miles, or to about ½ mile north of Patterson, the bottom still has the appearance of a timbered swamp, but this section is not boggy like the last, the soundings showing 3 to 5 feet of fine sand apparently overlying clay. Clay is exposed in creek bed.

At crossing of east and west road at Patterson, I found 5 feet of mud and then apparently coarse gravel. The United States Geological Survey Map is incorrect in showing a divide ½ mile north of this last point; the brook is continuous through this gap.

For the next 4 miles south the bottoms are from 1,500 to 2,500+ feet wide and generally

timbered and swampy in appearance. For 1 mile south of the Patterson road there seems to be generally from 3 to 4 feet of fine sand with clay bottom; for the next mile, or mile and a quarter, there is a pocket of 10+ feet of black mud.

For ½ mile north and south of the road crossing near Haviland hollow, the bottoms are high (3 or 4 feet above water line), dry and open; sand and clay 10 feet deep, then coarse gravel.

1 mile south of this latter road crossing or about 7½ miles from outlet dam, I again found 10+ feet of soft mud about 100 feet from east side of swamp.

At 1½ miles south the formation was 5 feet of fine sand over rather soft clayey bottom, and this continued to a point 8½ miles south of outlet dam, where the valley is narrowed to about 300 feet width by side hills of boulder drift. At this narrow place there may be some boulders found in the bottom, but the indications in bed of the creek pointed more to coarse gravel, with a good many cobbles, but not many boulders.

Southward from here nearly to the end of the tenth mile, it is open meadow, 2 or 3 feet higher than the river, and generally fine sand about 5 to 6 feet deep overlying clay, though I found 10+ feet of mud at one place about ½ mile north of the road.

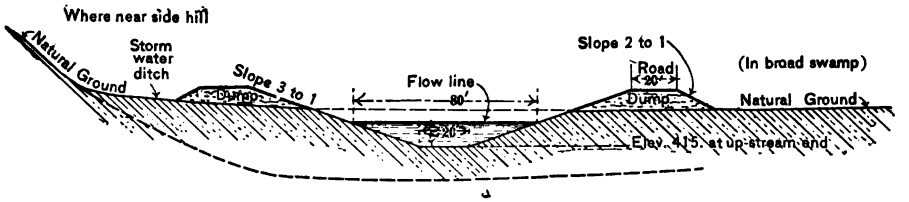
At the road crossing, boulder drift encroaches on the valley again, and also again at the end of mile 10.

These intrusions of boulder drift would in each case not affect more than 400 to 500 feet in length of the canal, and are at no place of such nature as to prohibit the use of dredges.

It appears most improbable that ledge would be encountered anywhere along the line of this proposed canal, at any depth of less than 15 feet, unless it may be in the first ½ mile through Pawling, and even there I am strongly of the opinion that there is generally 10 feet or more of drift overlying it. The general character of the ground appears favorable for excavation by a dredge for substantially the entire length of the proposed canal."

In our estimates we were further guided by the height of ground shown by the contours of the Worthen Map of the Croton Watershed, and by a number of elevations of ground along the bed and banks of the East Branch of the Croton river, kindly given by Col. Meehan from the results of surveys made under his direction during the past season. The ground is so remarkably level that the resulting estimate of quantity of material to be moved cannot be far in error.

If this 150 million gallons of Ten Mile river water were simply let run from Pawling Outlet Dam without any new open canal being excavated, it would find its way along present water-courses into East Branch Reservoir, but there would be claims for damages in making certain meadows more wet, and the railroad culverts are insufficient to pass large volumes. Apart from these reasons I believe a deep open canal for all, or nearly all this distance, well worth its cost as a means of draining the swamp and improving quality of water, although only the Ten Mile river were taken; with the upper Housatonic added this canal becomes an absolute necessity.



Open Canal through ancient River bed and Swamp south from Outlet Dam at Pawling. **Fig. 85**

The above width is for Ten Mile only, if Housatonic is added, the bottom should be made 20 ft. wider.

The form of channel proposed is shown in Figure 85. This form of section is designed more particularly for the portion through Pawling Village and for a mile or more below it. The excavated material is to be placed so as to form embankments or dikes on either side of the canal, and with channels for storm water outside these dikes, by which the street wash during a storm, and any chance surface drainage of Pawling Village would be excluded until it had run down a mile or more to a point where it could be ponded, and then let into the channel only by natural, slow, sand filtration. Where the canal

runs through muck it is proposed to cover the bed and banks with a thick layer of gravel.

The details of carrying floods of the East Branch of the Croton river without wash of the banks of this canal can be arranged later without noteworthy addition to expense.

The brook now entering the head of Swamp river from the west, south of Pawling Dam, would be diverted to above the dam, aad the cost of this is covered in the estimate.

Eleven and five-tenth miles open canal of the general section shown in Figure 85, bottom slope 5 inches per mile, bottom width 10 feet for upper 2 miles, 20 feet bottom width for lower 10 miles, side slopes, 3 to 1, graveled.

Estimated earth excavation, 2,200,000 cubic yards, at 25 cents, including all rock, boulders and hard gravel..... $550,000

Surface indications strongly indicate this material could be very cheaply excavated and deposited by a floating dredge, of either the centrifugal or endless chain and bucket type, at half this price.

The geological indications are that this channel, so far as line was reconnoitered, lies wholly in the bed of a broad ancient river of small declivity, which has become diverted and its bed deeply filled with water-borne deposits of fine sand, and that little or no ledge would be encountered.

This $550,000 for 11.5 miles averages $9 per lineal foot. A canal of same type, shown in Figure 58, and about the same size of cross-section 3 miles in length, in material probably more difficult, excavated by hand on the Massachusetts Metropolitan Works in 1897, actually cost about $10 per lineal foot.

Land damages along canal—

Say 2,000 acres wet swamp, averaging at $25.....	$50,000	
" 200 " grass land, at $100.....	20,000	
		$70,000

The land through which this canal would run is mostly a wet swamp, averaging about ½ mile wide, of almost no value in the middle half, but with some good grass land along the outer and drier portions. It would probably be best in the long run for the city to buy outright all of the swamp land at the outset and control it, and convert it by drainage into good grass land, which would improve quality of water collecting in Sodom Reservoir.

SUMMARY—TEN MILE WATER SUPPLY RESERVOIR AND CANAL.

Dam ¾ mile above Webatuck.....	$1,605,000
Land and dwellings on reservoir site.....	2,307,000
Probable damages for taking of mills and all water-power sites in basin; possible damages for diversion of water-power from Housatonic; possible damages for impairment of established business	900,000

Removal of cemeteries	$80,000
Removal and grubbing of trees, stumps and bushes; removing mud from mill ponds and clearing up portion of basin not stripped.	135,000
Removal of turf, muck and loam from entire southeast arm of basin from Dover Furnace to Pawling, 10 miles in length, 3,100 acres	2,170,000
Change in location of Harlem R. R., about 17 miles at $59,000.	1,000,000
35 miles new highways and 3 new highway bridges across arms of reservoirs	875,000
Outlet dam and sluices at Pawling	135,000
Deep channel for drawing reservoir low, sufficient for taking Housatonic water also if wanted in future	200,000
Storm water channels and filtration dam and diversion of two brooks in Pawling	50,000
New deep channel 11.5 miles long draining swamp and delivering water into Sodom Reservoir (not absolutely necessary for entire length, and can be extended 10 years hence as cheaply as now)	550,000
Purchase of swamp lands along East Branch of Croton River (say)	70,000
Sewage disposal works for Pawling and Patterson (say)	100,000
Total	$10,177,000
Add for engineering and contingencies at 15 per cent., instead of the ordinary 10 per cent., because of lack of precise surveys, 15 per cent. on $10,177,000	1,526,000
Add for contingencies on real estate and other settlements, say.	1,000,000
Total estimated cost for supply of 150 million gallons per day delivered into Sodom Reservoir at 415 feet elevation above sea level, or into Croton Lake and Aqueducts without additional expense ($84,654 per million gallons)	$12,703,000

For comparison we note that the estimated expenditure for an additional supply of 111 million gallons per day to Massachusetts Metropolitan District, now under construction—for reservoir (including complete stripping of bottom), lands, buildings, water rights taken, cost of dams and aqueduct as far as Sudbury Reservoir, was $11,883,000

This last amount corresponds closely to cost of works estimated above, and tends to show that the cost of Ten Mile supply is not unreasonable for the quantity of water to be obtained.

Appendix No. 7.

SUPPLY FROM THE UPPER HOUSATONIC.

The Housatonic river, as a source of supply for New York, was investigated by Horace Loomis, C. E., in 1878 and 1879. A report dated April 21, 1879, giving a full description of the works proposed, will be found in the Report of the Department of Public Works for 1879. This plan was radically different from that herein proposed and on a very much smaller scale.

It was then sought to take only 100 million gallons per day from the ordinary flow of the Housatonic without any special provision for storage, and to deliver this by means of a tunnel and open canal along the contour of the hillsides into the Croton Watershed at Pawling. Gaugings of the main stream were made, and the minimum yield without the construction of storage found to be only about 170 million gallons per 24 hours. It had been at first proposed to take the Housatonic at Bull's Bridge, and utilize the surplus flow above the 100 million gallons for elevating the supply sufficiently so that it would flow by gravity toward the Croton, but the low water flow of the river was found so small as to make this out of the question.

The most economical of the remaining projects considered by Mr. Loomis was that of tapping the river at West Cornwall and conducting the water by gravity to Pawling. It is of interest in this connection to note that Mr. Loomis, after examining the East Branch of the Croton, estimated that the sum of $81,312 expended for dredging, removing certain reefs and ledges of hard pan, would deepen the channel enough so that this 100 million gallons per day could be safely conveyed down the river chanel past Patterson and beyond. It will be noted that I have estimated upon a very much more liberal basis for the improvement of this channel.

It is also of interest to note that Mr. Loomis estimated the damages for developing 100 million gallons per day at $185,000 to be paid the proprietors of the water-power at Bull's Bridge and the other mill sites.

Hardness of Ten Mile and Housatonic Water.

Another item of Mr. Loomis's report, of special interest in considering the present plans, is a chemical analysis of the water by Professor Charles F. Chandler, who reported it very much the same as the Croton water, and with hardness only one degree greater than the Croton. This question of the softness or hardness of the Housatonic water gave me some concern, because

of a large amount of lime-stone formation shown by the geological maps along its head waters, and I had one or two samples carefully collected and analyzed. The hardness of these samples was found greater than the best practice in New England commends, but this test was manifestly unfair because the samples were taken after a drought of unusual severity, described by the farmers along the Ten Mile and the Housatonic as "the worst that they had known for twenty years." The water flowing in the stream at the time the samples were taken was almost entirely ground water, and not in any way fairly representative of the average flow of the year, or of the spring flood water as it would be found after storage in a great reservoir. From considering the matter with some care, I am led to believe that the hardness of the water would be a little greater than that of the Croton, but not sufficiently so to cause special comment or trouble, or to interfere with the ordinary uses for the laundry and the generation of steam, but I would earnestly recommend that the collecting of weekly samples be at once begun and continued through at least one year, and that note be made of the quantity flowing at the time of each sampling, and each sample be multiplied by a factor proportional to the volume of flow, when making up the average hardness for the year.

Possible Pollution of Housatonic.

The Upper Housatonic is mainly a mountain stream, draining the Berkshires, but on its banks are the towns of Great Barrington and Pittsfield, a long way upstream. I am informed that the purification of the sewage of each of these towns before permitting it to enter the stream is now under active consideration, and under the excellent practice of the Massachusetts State Board of Health there appears to be very little doubt that this objectionable element will soon be removed in large degree.

A very great safeguard to the healthfulness of the water delivered from the proposed Ten Mile and Housatonic Reservoir is found in the great size of this reservoir, the long period of subsidence, and in the peculiar shape of the reservoir (see Figure 16), by which any given mass of water would be so long in traveling from the inlet to the outlet of the reservoir, that the harmfulness of any pathogenic bacteria would probably cease. A good illustration bearing upon this is the experience with the Mystic Reservoir of Boston, which, with the advent of the new Metropolitan supply, has been abandoned, but which was used for many years to supply a large portion of the community. The small river that fed this flowed through a very populous district and had many sources of pollution; as I had frequent occasion to

observe. The reservoir forunately was of such conformation that the water was a long time passing from the mouth of the river to the intake of the pumps, and no other good reason appears for the actual absence of epidemics that might have been expected from the impure water flowing into the reservoir.

Stripping Reservoir Bottom.

I have entered in the estimate of cost a very large sum (more than two million dollars) for the complete stripping of the southwesterly arm of the basin for its entire width and for a length of upward of ten miles, and were it not for the statements of Mr. Birdsall and Mr. Fteley, that the Croton Reservoirs have, in their opinion, shown little need for any such artificial removal of the organic matter from their bottoms, I should have gone further in this estimate and provided for stripping a still larger area. It will be noted that *the central portion of the basin for which no estimate for stripping has been included is much of it so deep that the troubles sometimes incident to shallow reservoirs having organic matter at the bottom would be probably avoided.*

None of the Croton Reservoirs have had the turf and organic matter removed in this manner, yet it is stated that after a few years of use they come into satisfactory condition, but in my various inspections of the Croton Lake in particular, I have at times seen such growths of algæ as to prompt a recommendation for stripping the area already mentioned, notwithstanding its large expense, and in view of the careful studies of the Massachusetts State Board of Health and the engineers of the Boston Water Board and the conclusions reached regarding the recent reservoirs for Boston and the Metropolitan supply *I would strongly recommend that this matter of the possible advantage of stripping, at least the shallow portions (over which water will ever be drawn down to a depth of less than, say, 10 or 20 feet), for all future reservoirs for New York should receive further expert study,* unless, indeed, filtration is to be planned from the first.

With this stripping of the lower twelve miles of the Ten Mile Reservoir down to clean gravel (or covering with gravel the portions not so stripped), with the very ample marginal strips provided for in my estimate of cost, with the covering of bog holes and old mill ponds with a thick layer of clean gravel, there can be no reasonable doubt that this Ten Mile or Housatonic water will be as good, or better than the Croton.

FUTURE FILTRATION.

In considering a comprehensive system of water supply for the future, it is well to ask if the tendency must not everywhere be toward filtration, and if the public will not, sooner or later, become educated to a more exacting standard, and to demand a water more clear in its appearance and more attractive in taste at all seasons of the year. The cost to the average citizen, man, woman and child in New York, is now only a dollar per capita for his domestic water supply, and it may well be questioned if he will not at some future time, and perhaps not distant, willing pay, say, thirty-five cents* more per year per head, or $3.50 per year for a family of ten, to have this supply filtered. Whenever the public becomes educated up to demanding restriction of waste and to consent to universal metering, then this ideal of filtration will be much more easily and cheaply attained. The question therefore comes, is it not wise to now, when planning aqueducts, to arrange the grades so filtration can be cheaply applied at any future time without pumping? And when planning a reservoir like the proposed terminal reservoir at Park Hill, or the reservoir at Rye, is it not best to secure a generous tract of land which would be available for the future filtration plant, and could in the meantime, if not needed for filter beds, serve admirably as a park?

The water system could readily carry the expense. Personally, I firmly believe that such is the only wise course, and that provision for obtaining suitable area and provision for a drop of, say, 10 feet are sufficient for manipulation of filters without expense of pumping should be incorporated in the plans for the new supply which must soon be constructed.

Attention may also be directed to the remarkable stretch of level land extending for twelve miles along the side of the proposed channel between Pawling and Milltown Bridge. It is not impossible that under the rapid development of the art of filtration, some portion of this could be availed of for natural filtration.

I have hesitated to include any estimate for filtration in the cost of these proposed plans for the Housatonic water merely because it is at variance with ordinary practice, but let there be 10 to 20 years of earnest work in stopping waste and enough could be saved to go far toward providing filtration for every gallon of water delivered and with the available pressure and volume give New York throughout its several boroughs in truth the best large water supply in the world and all at a surprisingly low cost.

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* 35 cents per capita for 2,000,000 inhabitants — $700,000 per year, at 226 million gallons per day total consumption and waste in 1899 × 365 days — 82,600 million gallons per year, or 35 cents per capita is $\frac{700000}{82600}$ — $8.50 per million gallons for filtering everything for Manhattan and Bronx, including waste.

HOUSATONIC DAM SITE.

Nature has favored the Housatonic project by providing a deep, narrow gorge, with granite walls, where one dam of almost precisely the same size and height of the New Croton Dam now under construction, will hold all the water of a thousand square miles of watershed, of both the Housatonic and the Ten Mile rivers in storage and turn it into the Croton Watershed at Pawling. In Figure No. 79 the transverse section of the valley at this point about one mile below Merwinsville, Conn., is shown and, for comparison, a section of the Croton Dam site measured out to the actual lines of construction, is also shown in the same drawing, superimposed.

The Housatonic Dam site appears, however, very much more advantageous than the Croton because of it being in sound granite, instead of bedded in a rock like the partially disintegrated limestone on which a large portion of the Cornell Dam rests. I have not had opportunity to make borings in the bed of the stream, but the ledge is plainly exposed at many points, as indicated in the plan, Figures Nos. 81 and 82, by the cross hatching. On the easterly side of the valley, in particular, the covering of drift gravel on top of the granite is very shallow, giving abundant opportunity to examine its characteristics. The question may naturally arise as to whether this deep, narrow gorge had its origin in a fault or displacement of the bed rock and whether there is likely to be a line of seamy, blocky or partially disintegrated stone found at the bottom of the valley. I therefore called upon Professor William O. Crosby, of the Department of Geology at the Massachusetts Institute of Technology, and Consulting Geologist to the Massachusetts Metropolitan Water Board, under whose supervision much of the work of exploration by diamond drill borings has been made for the dam site at Clinton, to make an examination of the surroundings of this Housatonic dam site.

Professor Crosby's report on this dam site will be found in Appendix No. 16, page 585.

The opinion is given by Professor Crosby that in all probability sound rock would be found at the bottom of the gorge. The indications of the surroundings are extremely favorable as to the homogeneity of the rock on the two sides of the valley and to the valley having been caused simply by erosion. The outcrop of the east side is particularly sound and smooth, evidently made so by glacial action, and the probabilities are that under the shallow covering of modified drift in the lower half of the section, the rock will be found even more sound than at the outcrop, because of this drift deposit having shielded it from the action of frost. The stone at both ends of the dam site appears admirably adapted for the construction of the dam, and the sur-

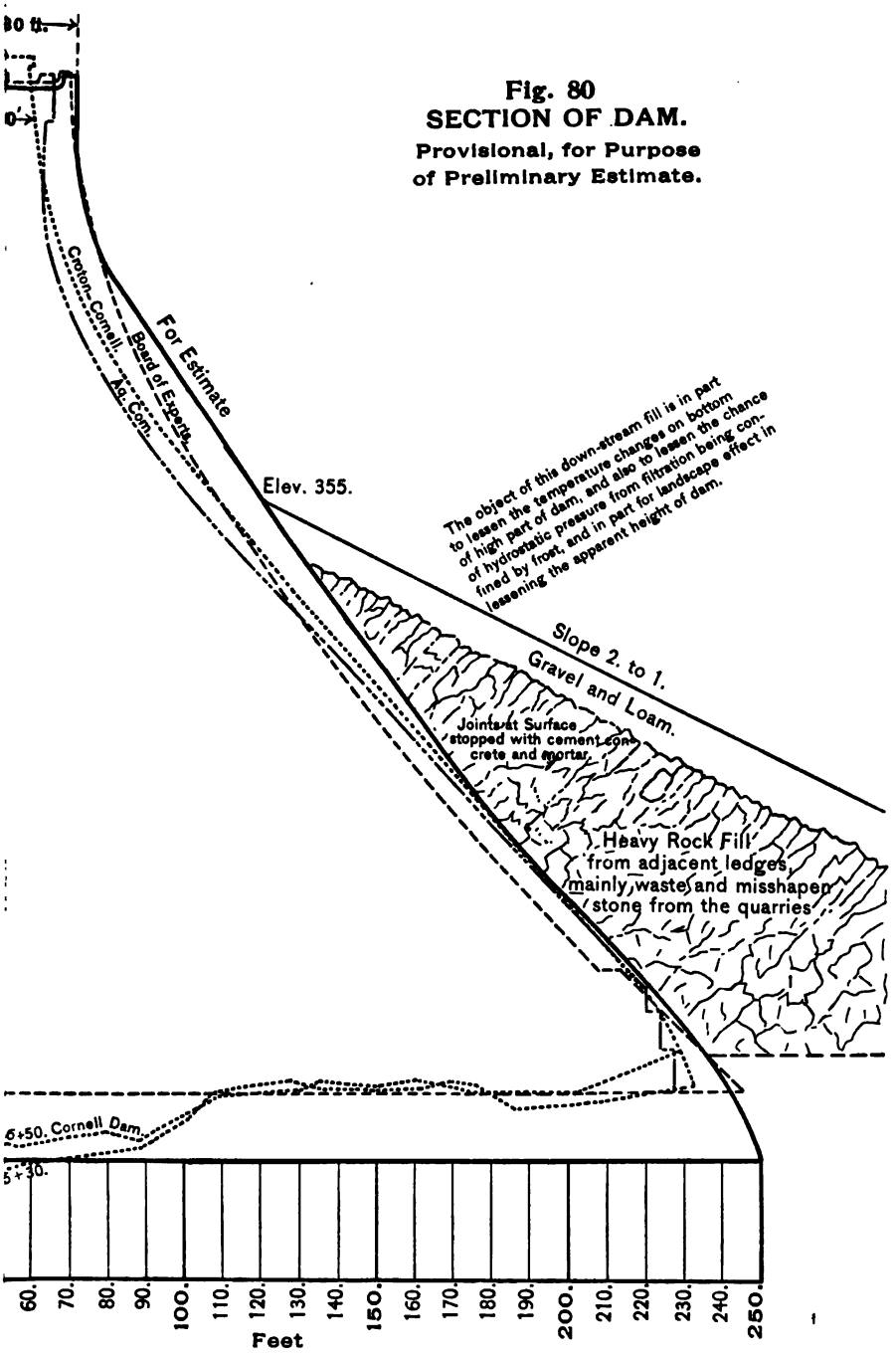

Croton-Cornell
Board of Experts
Aq. Com.
For Estimate
Elev. 355.
The object of this down-stream fill is in part to lessen the temperature changes on bottom of high part of dam, and also to lessen the chance of hydrostatic pressure from filtration being confined by frost, and in part for landscape effect in lessening the apparent height of dam.
Slope 2. to 1.
Gravel and Loam.
Joints at Surface stopped with cement concrete and mortar.
Heavy Rock Fill from adjacent ledges, mainly waste and misshapen stone from the quarries.
6+50. Cornell Dam.
5+30.
60. 70. 80. 90. 100. 110. 120. 130. 140. 150. 160. 170. 180. 190. 200. 210. 220. 230. 240. 250.
Feet

Fig. 80
SECTION OF DAM.
Provisional, for Purpose of Preliminary Estimate.

Proposed Housatonic Dam (for purpose of estimate) is shown ——————
New Croton Dam as built, Cornell site " ----------
" " " Quaker Bridge by Board of Experts " – – – – – –
" " " " " " Aqueduct Com'rs " ——— · ———

face indications all point to the feasibility of opening an excellent quarry at each end from which a single reach of cable-way could land the blocks of stone on the dam.

Time of Building.

The completion of this dam fixes the date when Housatonic water could be delivered in New York *via* Croton Aqueducts.

This Housatonic dam could, without doubt, be built more quickly than the New Croton Dam. Much time was lost in getting started on the New Croton Dam, and the enormous amount of excavation required to reach sound rock also added much to the delay. Three and a half years* were spent in getting down to sound rock and in preparing the bed before the laying of the foundation was begun. The opinion is held, if I mistake not, by the engineers in charge of the construction of the New Croton Dam, that if the work was to be done over again, under the experience gained, that dam could be completed within seven years. This work on the Croton has been carried along summer and winter, but has not been pushed very hard in cold weather. No night work in laying stone has been attempted.

It would appear not outside of the possibilities, in view of the very favorable location of this proposed Housatonic dam in relation to its quarry site, the small amount of excavation in cleaning off the rock preparatory to laying stone, and the possibility of carrying on the work throughout the twenty-four hours, by three shifts of workmen, that this dam could be constructed in five years from the time work was begun.

Question will. of course, at once be raised as to the expediency of night work, with the danger that under poor illumination, less perfect inspection and the confusion of shadows, some cavities may not be properly filled.

Difficult and important work, subject to great pressure, is carried on in the darkness in tunnels and in caissons under many disadavantages and the possibilty of securing sound mason-work at night would appear to be largely a question of organization and plant. Methods of illumination have greatly improved in recent years, and, if need be, a larger factor of safety could be provided,** by thickening the structure.

* See Trans. Am. Soc. C. E., Jan., 1900. C. S. Gowen on Croton Dam. Work begun Oct., 1892. Foundation of Dam begun May 28, 1896. Completion expected in 1902.

** A possible method of guarding against penetration of water that is so radically novel that I hesitate to suggest it, is that of insuring the imperviousness of the dam to infiltration pressures, by providing a quarter-inch continuous sheet of lead, placed vertically, say in a plane some 10 feet back from the top of upstream edge, and with the mason-work on the two sides firmly bonded by bars of railroad iron passing through the lead.

From the studies of a very distinguished engineer, who has recently inspected many masonry dams with a view to this transmission of hydrostatic pressure by infiltration, I am led to think that this or some other device for insuring the freedom of the interiors of the masonry from hydrostatic pressure when frost seals the exposed surface is worth serious investigation.

All of these details are matters of the future. To make the estimate a safe one, I have based it upon a curved or arched form for the dam (for which form I confess a partiality), and in order to get an outside figure, because the larger volume of masonry costs some $600,000 more than for a straight dam of the form now building on the Croton, and, as will be seen by the section of Figure No. 80, I have allowed for a very much greater thickness of masonry in the upper portion, for the purpose of making the estimate a safe one, however cautious the ultimate designer may be in providing against ice thrust.

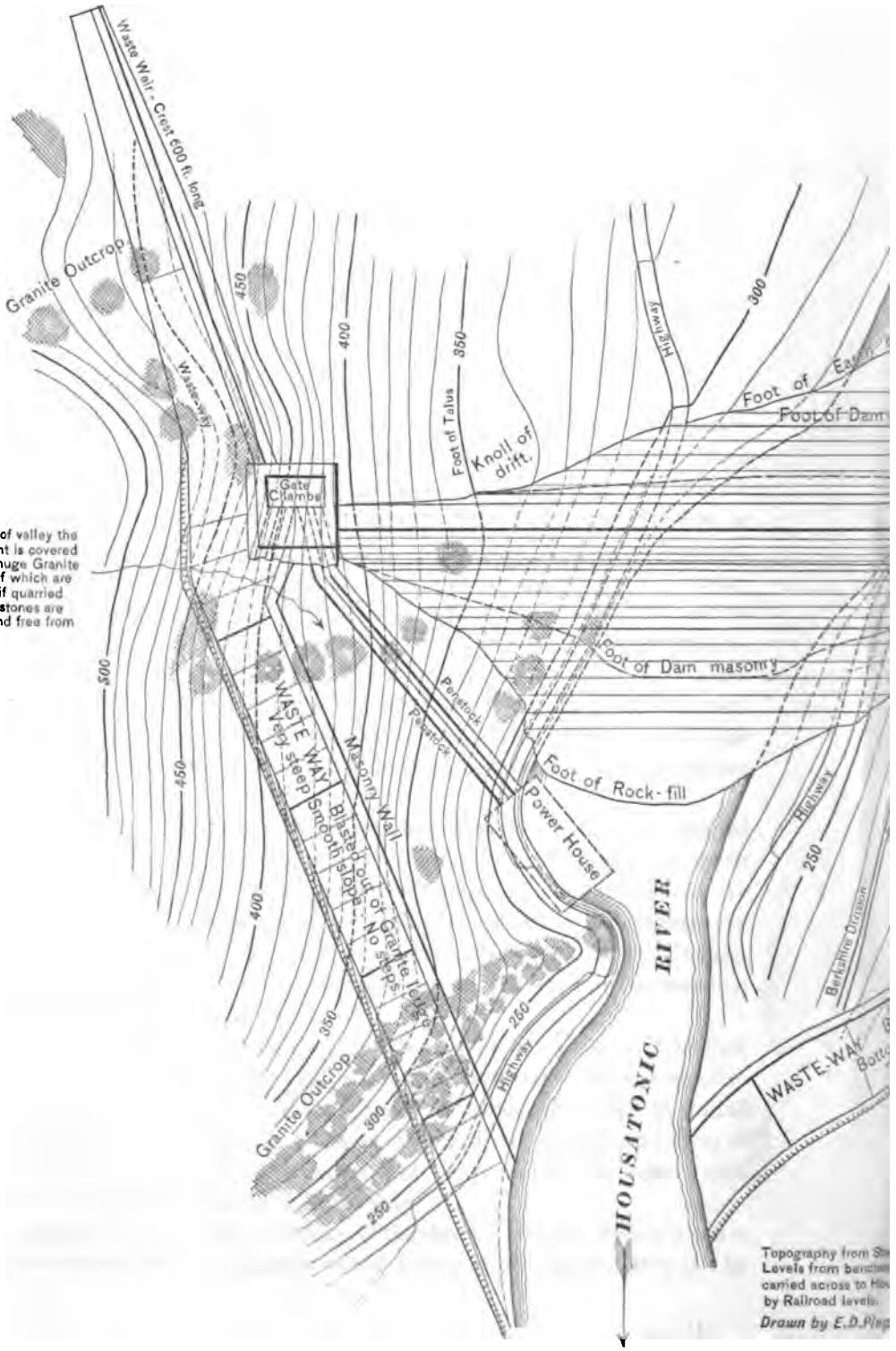
Elevation of Delivery of Water.

The elevation of the crest of the spillway is fixed by the storage volume required to fully develop the flow of the two streams at about 450 feet above the sea. By making it 5 feet or 10 feet higher the increase of cost would be relatively small and the safe yield of the stream could be forced up to 800 or 850 million gallons per day instead of 750. Particularly if filtration becomes the rule by the time this large flow is needed. Nature, in forming the topography of the basin, has fixed the height of the flow line at a level of 420 to 450 feet above sea, and having the water stored at this level, it is but natural and reasonable to avail ourselves of this elevation in providing New York City with a water supply at greater pressure than it has heretofore enjoyed. If this pressure is to be wasted, it may as well be wasted, until wanted, near New York City, as to waste in running down unnecessary declivities on the way to New York.

The high pressure pumps of the city are already seriously overloaded and the expense for maintaining them is very large and appropriations of nearly $400,000 for additional high-service pumps and appurtenances are now being urged. The consumption of high-pressure water is growing very rapidly and it would appear unwise not to so plan new works that this Housatonic water or other water from the north can be delivered at the terminal reservoir at 300 feet above sea level.

The high service can be taken from this. One or two special 48-inch mains for a high pressure fire supply within the business district could also be taken from this reservoir, and ultimately I believe it would be found advisable, as new mains were laid and new service pipes put in, to plan them so that they would be safe against a pressure of 100 pounds to the square inch throughout all the lower parts of Manhattan.

It appears to me that the tendency of the times is steadily toward a water pressure of from 80 to 100 pounds to the square inch wherever this can be



On this side of valley the hillside at this point is covered by a heavy talus of huge Granite blocks, the angles of which are almost as sharp as if quarried yesterday and the stones are remarkably sound and free from seams.

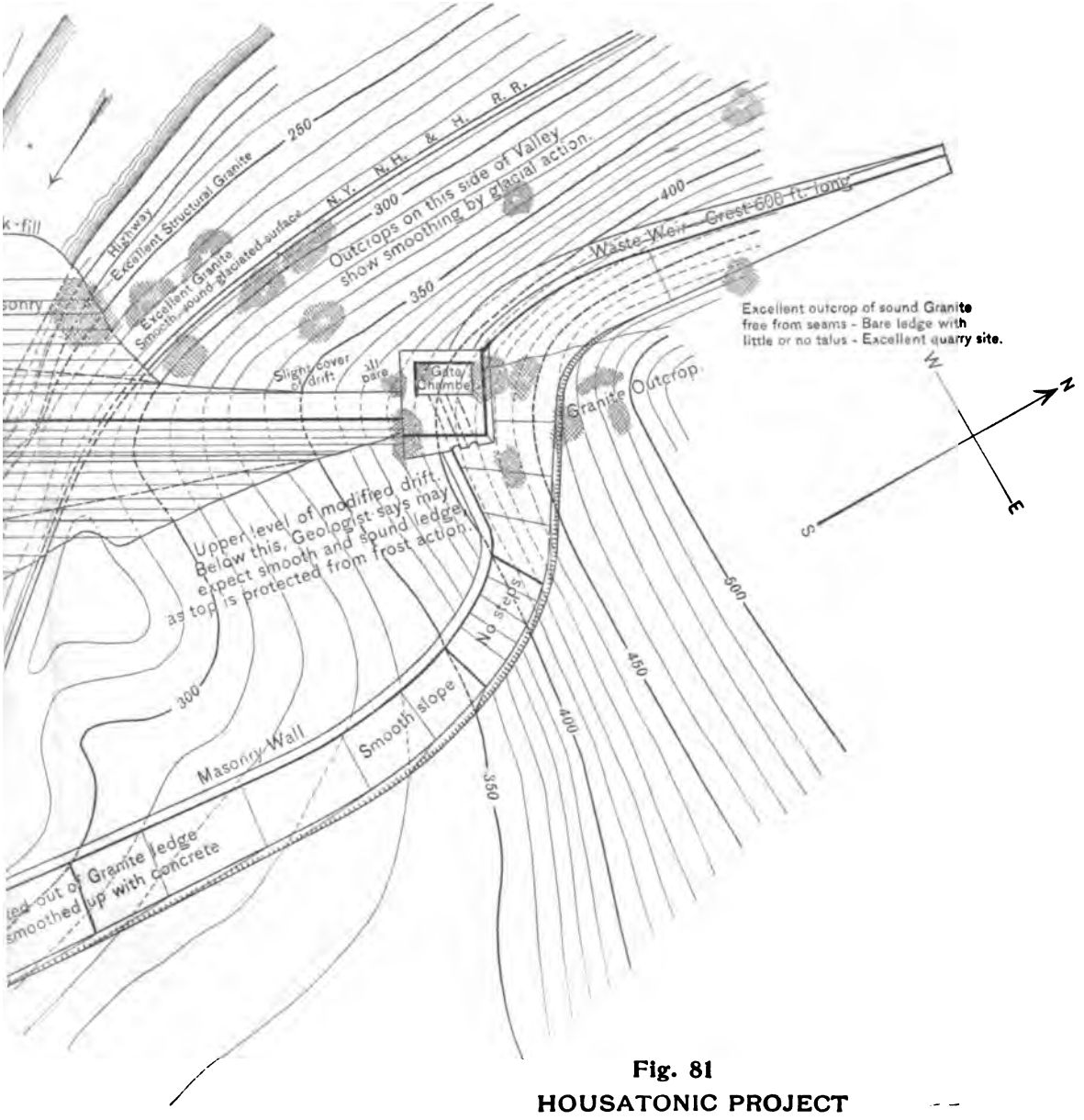


Fig. 81

HOUSATONIC PROJECT

DAM SITE ABOUT ONE MILE BELOW MERWINSVILLE

WITH STRAIGHT MASONRY DAM

Scale

0 50 100 ft. 200 ft. 300 ft.

Surveys by Geo. A. Taber, C.E. October. 1899.
New York City Dep't of Water Supply in Croton Basin
Hic R.R. (Berkshire Div. N.Y. N H. & H. R.R.) and verified

obtained at moderate expense. The City of Syracuse is an interesting example, and its recent experience does much to disprove the statements that have often been made, based on theory and not on practice, about the greatly increased waste which comes with a high pressure supply and about the necessity of renewing all of a city's mains and service pipes whenever the pressure is so increased. In Syracuse the pressure was increased 50 pounds or from 45 to 95 pounds per square inch; no noteworthy increase in draft is said to have followed, very few changes were found to be required in the street mains and very little difficulty with the house plumbing. The engineer under whom these great improvements in the Syracuse water supply were carried out, is now the Chief Engineer of the Croton Aqueduct Commission.

The two cities in the United States where waste of water is the least (because of meters) *are cities having very large areas in which the pressure is from 80 to 120 pounds per square inch.*

There is much experience to sustain the view that there is less waste under the high pressure because of the escape being more quickly noticed and more noisy and so more promptly remedied. The aid to fire protection from this high pressure and, ultimately, the saving in cost to the city in insurance rates, which must always ultimately be adjusted in fair ratio to the loss, would be very large; and the saving in maintenance and extension of fire department service would also be large.

The automatic sprinkler is the most effective and most economical method of fire proofing known and is the only possible method that can be applied to large department stores, factories having inflammable contents and large areas and the like. The efficiency of a sprinkler system is at its best only when fed by a high-pressure, continuous, gravity supply. Some years ago two of the most prominent underwriters in Boston, one of them the late John C. Page, expressed to me their belief that if Boston could have a pressure of 100 pounds to the square inch on every hydrant, this would ultimately mean a saving of upwards of $100,000 per year in insurance premiums apart from any question of sprinklers and the like. Of course, such statements are of necessity mere matters of opinion, and other very able underwriters may hold different opinions, but from having had occasion to study fire protection carefully for twelve years past, I have come to have a high appreciation of the value of a high pressure gravity supply and would most strongly urge that this natural elevation of the Ten Mile and Housatonic sources—if these are secured—*be availed of for a high-pressure supply in the city instead of being used up in unnecessarily high declivity in the conduit lines.*

ESTIMATED YIELD OF UPPER HOUSATONIC RIVER WITH DAM ONE MILE BELOW MERWINSVILLE (including the Ten Mile River).

Total area of watershed (by planimeter from maps U. S. Geological Survey—scale 1/62500).....	1,020 square miles.
Area of water surface in proposed reservoir with flow line at elevation 455 feet above sea..... (This is 5 feet greater elevation than proposed for the Ten Mile River Dam.)	33 square miles.
Probable additional evaporating surface in drainage area outside of reservoir limits, consisting of ponds, streams, and one-half the area of swamps.	10 square miles.
Average area of reservoir as drawn under conditions of full use (about equivalent to area at elevation 448), about.....	30 square miles.
Total average extent of water surfaces in watershed with reservoir at probable average levels required in maximum use.....	40 square miles.
Net land surface in watershed, from map.....	980 square miles.
Per cent. of water surface in entire watershed to land surface	4.1 per cent.
Storage volume between elevation 455 and elevation 425 (volume computed from contours on U. S. Geological maps in part readjusted to our levels.)	180 billion gallons.
Storage volume per square mile of net land surface between elevation 455 and elevation 425.....	184 million gallons.
Probably detailed surveys will show it feasible to draw this reservoir down to elevation 420 in extreme drought, thus increasing the available storage 5 feet in depth over 20 square miles, making 20 billion gallons increase of storage. The storage can be largely increased by building the dam 5 feet higher, making total available storage upward of.....	200 billion gallons.
By Diagram No. 48 of yield of watersheds, as derived from Croton data, a watershed with 4 per cent. water surface and 200 million gallons of storage per square mile of gross area may be relied on for a safe yield in the most severe low rainfall cycle yet known on the Croton of 865,000 gallons per square mile of gross watershed per day. 865,000 × 1,020.....	882 million gallons safe yield per day.
Housatonic safe yield on basis of Croton Records.	
(The Croton Watershed in 1902, without flash boards, will have 193 million gallons of storage per square mile, which is, in part, less advantageously located than this Ten Mile and Housatonic Reservoir, being near heads of tributaries.	
The yield of 275 million gallons adopted for the Croton is 763,000 gallons per square mile of gross area per day, and 800,000 gallons per square mile of net land per day. The per cent. of water surface is the same on Croton and Housatonic. Therefore, call the yield from the Housatonic the same.) 800,000 × 980 =	
Housatonic safe yield on basis of 275 Mil. Gal. from Croton.....	784 million gallons safe yield per day.

Greatest length of time that reservoir level would have ever been continuously below level of spillway in past 32 years with this rate of uniform draft according to Croton records..... 3 years.
(May 15, 1870, to February 1, 1873.)

Maximum uniform rate of draft per square mile possible with 184,000,000 gallons storage per square mile and water surface of 4.1 per cent. of whole watershed exposed to evaporation—according to Sudbury records for past 26 years, which, per Boston rainfall records, includes longest severe drought of the past half century, but with no one year so dry as the rainfall records indicate for 1826. See Diagram No. 49..... 750,000 gallons.

Housatonic safe yield on basis Sudbury Records,
750,000 × 980 =..... **735** million gallons safe yield per day.

Greatest length of time that reservoir level would have been continuously below spillway in past 25 years at this rate of draft, per Sudbury records (March, 1880, to February, 1883, or possibly from January, 1879, to December, 1883)..... 3 to 5 years.

Rate of uniform draft per square mile of net land surface adopted by J. R. Freeman as to be relied upon under the most severe drought yet recorded and as best adapted to utilize resources of this reservoir without impairment from weeds growing on margins, etc., and by flowing reservoir to elevation 455 and drawing to 425 only..... 765,000 gallons.

It appears safe from Croton rainfall records to assume that rainfall and yield on these southern slopes of the Berkshire and Housatonic Mountains is larger and evaporation from swamps less than on the Sudbury.

Safe yield as adopted by J. R. Freeman in million gallons per 24-hour day from this watershed of 1,020 square miles, of which 980 square miles is land and 40 square miles water, corresponding to above figure of 765,000 gallons per day per square mile net land, with 184 million gallons storage per square mile..... 750 million gallons.

As shown on previous page, Croton records indicate a safe yield of fully 800 million gallons. To be conservative in absence of long gaugings and rain gauges for this particular watershed, I call it..... 750 million gallons.

This can probably be increased 50 million gallons, or to 800 million gallons per day, either by building the dam 3 feet higher or by drawing pond 5 feet lower, say once in 25 years; indeed the Croton data as per previous page show that with flow line at 455 this watershed would have yielded 882 million gallons in the most severe drought known since the Croton records began.

Probable average length of period in which water level in Housatonic Reservoir would be 5 feet or more down, when full flow of 750 million gallons per day is drawn (based on Croton records), about..... 8 months.

PROBABLE COST OF SUPPLY OF 750 (OR 800) MILLION GALLONS PER DAY FROM COMBINED UPPER HOUSATONIC AND TEN MILE RIVERS, WITH DAM ABOUT ONE MILE BELOW MERWINSVILLE, CONN.

Cost of Dam.

Dimensions for this estimate are based upon stadia surveys made under supervision of J. R. Freeman, October and November, 1899. Levels were checked from bench marks of Berkshire Division of N. Y., N. H. & H. R. R., across country to bench marks of the New York Department of Water Supply. We find excellent outcrops of sound granite ledge over the greater part of dam site, which leave little doubt as to limit of extra excavation below the natural surface. No borings were made, but the geological conditions for continuous clean, hard granite, without seams, over entire site, appears exceeding favorable. See Report of Geologist, Appendix No. 16, page 586.

Quarries of excellent building granite can be opened in hills above both ends of dam. One short stretch of cableway can lift granite out of quarry and place it on the dam. The railroad is within the limits of dam location, making it possible to deliver cement and other materials directly on the ground.

The quantities used in the estimate below are for the curved dam, in order to get a safe outside figure, because of this form requiring more masonry than the straight dam.

Curved Housatonic Dam—Preliminary Estimate from Careful Reconnaissance.

Cleaning off dam site—

Probable earth excavation, 60,000 cubic yards, at 50 cents..	$30,000
Rock " 70,000 cubic yards, at $2.....	140,000
Probable cost flume and temporary dams.....	500,000

Similar work for New Croton Dam cost about $350,000, but much of this work becomes available in the permanent structure, the cost of which is, however, figured in full without this deduction.

Facing masonry—range granite for both faces above earth fill, also for all faces of abutments—50,000 cubic yards, at $15.. 750,000

(Contract price on New Croton, $13.50.)

Rubble masonry, quarried about 1,000 feet distant, 870,000 cubic yards, at $4..... 3,480,000

Contract price on New Croton Dam, with quarry two miles distant, $4.05 per cubic yard.

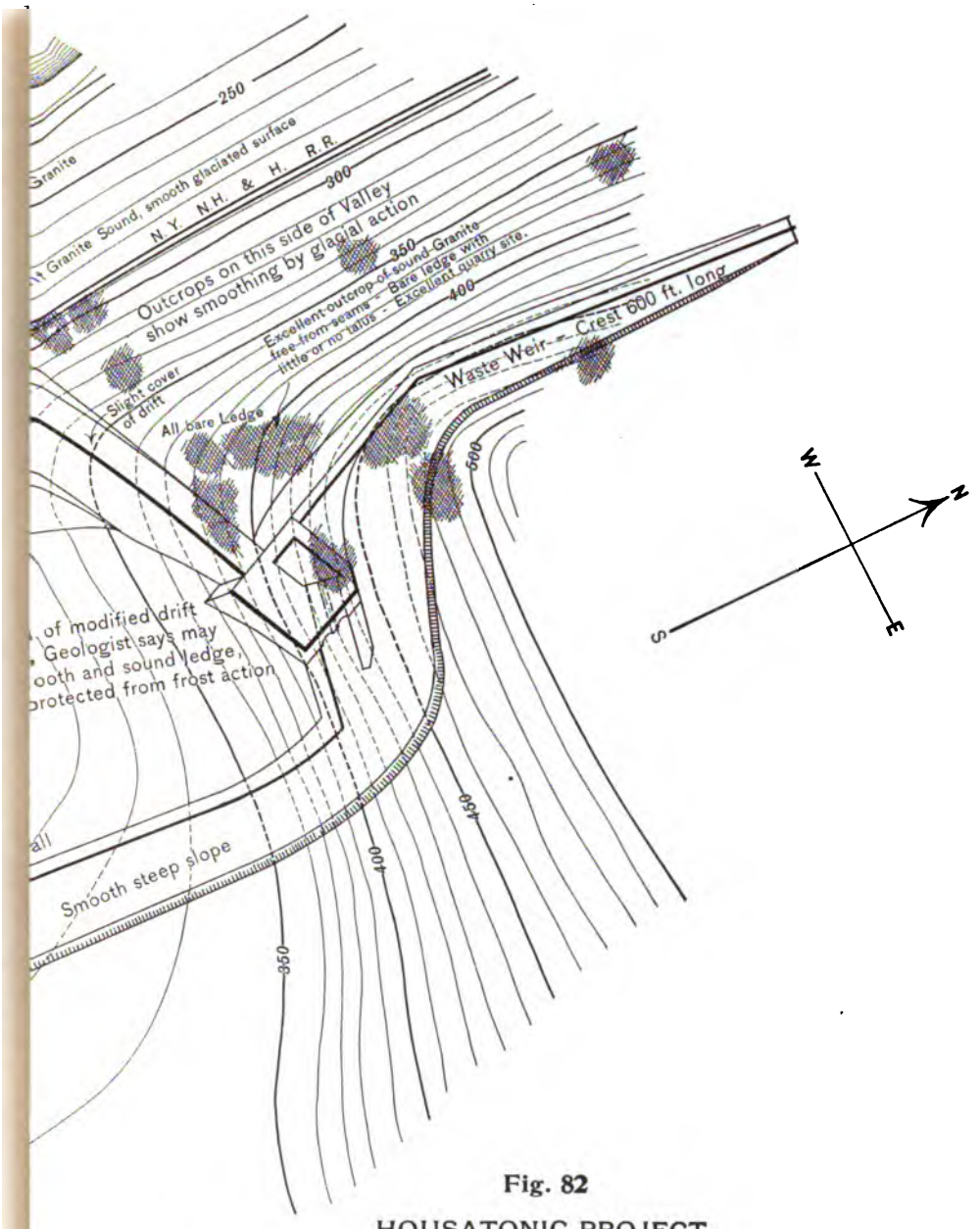


Fig. 82

HOUSATONIC PROJECT

DAM SITE ABOUT ONE MILE BELOW MERWINSVILLE

WITH CURVED MASONRY DAM

. October, 1899.
Supply in Croton Basin
.H. & H.R.R.) and verified

Scale

0 50 100 ft. 200 ft. 300 ft.

Dimension cut stone for 2 lines heavy coping, 2 heavy belt courses, gate-chambers and sluiceways, 7,000 cubic yards, at $25	$175,000
Massive dry rock fill, with earth top slope, as per Fig. 79, on downstream side, partly from quarry-waste and talus, 400,000 cubic yards, at $1.25	500,000
(Not strictly necessary to any such extent.)	
Earth fill on upstream side, as per plan, 200,000 cubic yards, at 50 cents	100,000
(In part would be made from preliminary excavation and coffer dam.)	
Two gate-houses, gates and apparatus, say	75,000
Probable total cost of curved dam and abutments	$5,750,000

Average—

$$\frac{\$5{,}750{,}000}{870{,}000 + 7{,}000 + 50{,}000 = 927{,}000 \text{ cu. yds.}} = \$6.20 \text{ per cu. yd.}$$

Titicus Dam, the largest yet completed in Croton Watershed, cost complete, per cubic yard $4.47

Cornell, at contract prices for dimensions of original estimate, would have cost, per cubic yard 6.76

—but this included very much more preliminary excavation and a quarry about 2 miles distant.

A straight dam at same location of same height and same cross-section contains about 100,000 cubic yards less masonry than the curved section and therefore probably would cost about $620,000 less than above.

Excavation for 2 spillway channels, as per plan, rectangular section, smooth, rapid slope, roughness leveled up with concrete, walled on river side.	
Rock excavation, about 65,000 cubic yards, at $1.54	100,000
Spillway, cut stone crests, 800 cubic yards, at $25	20,000
Spillway wing dams and wasteway walls, rubble in cement, say, 15,000 cubic yards, at $5	75,000
Portland cement concrete and rubble in cement for smoothing up waste-way channels, 4,000 cubic yards, at $6	24,000
Probable total cost of dam and accessories (not including contingencies and supervision)	$5,969,000

TABLE No. 51.

Land Damages.

Basis of estimate same as per statement on page 348.

Cultivated land, $100 per acre, including legal and other expenses. Buildings extra Pasture land, $30 per acre. From statements of actual sales these prices appear to be fully 50 per cent. above market value.

Marginal area outside flow line, 250 feet on steep hillsides, 500 feet on flat slopes, peninsulas and islands included.

See Table No. 50, page 349, for Land Damages for Ten Mile Basin only.

	LAND.	BUILDINGS.
Town of Dover, N. Y., below Webatuck Dam—		
Land, 5.3 square miles, or 3,390 acres	$303,000	.....
60 dwellings and other buildings	.....	$98,000
Town of Cornwall, Conn.—		
Land, 1.0 square mile, or 640 acres	55,000	.....
61 dwellings and other buildings	.....	100,000
Town of Sharon, Conn.—		
Land, 0.8 square mile, or 510 acres	42,000	.....
19 dwellings and other buildings	.....	35,000
Town of Kent, Conn.—		
Land, 11.8 square miles, or 7,550 acres	536,000	.....
225 dwellings and other buildings	.....	432,000
Town of New Milford, Conn.—		
Land, 5.4 square miles, or 3,460 acres	292,000	.....
97 dwellings and other buildings	.....	200,000
Town of Sherman, Conn.—		
Land, 5.0 square miles, or 3,200 acres	212,000	.....
28 dwellings and other buildings	.....	62,000
Total for 18,750 acres of land and 490 dwellings added below Webatuck Dam	$1,440,000	$927,000
Add for 15,300 acres of land and 513 dwellings previously estimated above Webatuck Dam	1,216,000	1,091,000
Total for 34,050 acres of land and 1,003 dwellings	$2,656,000	$2,018,000
		2,656,000
Grand total for value of land and buildings above Housatonic Dam, including the Ten Mile and Housatonic basins	.....	$4,674,000

The above includes marginal areas proposed to be taken for preservation of purity of water around entire circumference of the great reservoir. These were estimated as including all lands out to 250 feet outside the flow line on steep or wooded slopes, and all to 500 feet outside the flow line on slight slopes and cultivated land, and all islands and peninsulas projecting into reservoir were assumed to be taken and their area is included above.

No factories or mills on the main river (Housatonic) will be flowed out by the proposed dam, and only two mills on any of the branch streams; one of these is a small saw and grist mill on a branch of Womenscheunk Brook, the other a small cider-mill on Womenscheunk Brook.

The three improved water-powers at New Milford, Gaylordsville and Birmingham would be seriously affected as shown in Appendix No. 8, also the five projected water-power sites not yet improved, and the three abandoned water-power sites at Kent Furnace, Swift Bridge and Cornwall Bridge would have the water that flows past them much lessened in volume.

The real injury to all these water-powers and possible mill sites combined would probably not exceed $500,000, but to cover possible awards, legal expenses, expenses of experts, etc., etc., I have put into the estimate for possible damages if act is so drawn as to assume immediate and complete diversion $2,000,000

Water-power taken under eminent domain or purchased under public necessity appears to commonly bring a much higher price than its fair market value for purposes of use as shown by *bona fide* sales of similar property or previous sales of the same property. This possibility of excessive appraisal must be provided for in our estimate.

The difference between the real value of a water-power and its theoretic value as found by the common method of capitalizing the annual cost of providing an equivalent amount of steam-power, comes from certain defects and inconveniences of a variable water-power on a small stream which, although clearly recognized by the manufacturer and the purchaser intending use, cannot be clearly set forth in figures, while on the other hand the problem presented by ingenious counsel of so many horse-power to be replaced by steam at so much per day, appears straightforward and simple. This does not tell the whole story and with the proper sympathy for the party whose property is forcibly taken he commonly gets the benefits of all doubts.

I have therefore in these estimates been guided more by certain settlements for actual large diversions of water-power taken away for public purposes than by my opinions of the fair market value to an intending purchaser.

As already stated, it is entirely practicable and inexpensive to regulate the outflow of water stored in the spring floods in this enormous reservoir, all of which will not be needed for New York for many years in the future, by letting a part of this out in the dry summer weather.

With this policy followed, *the water-power at all mill sites on the Housatonic below Merwinsville will not be injured in the slightest degree for fully fifteen years to come*, and instead of an injury, each mill site would receive for this time (until 1915) great benefit in the lessening of spring floods and the

assurance that in severe droughts a much larger volume would flow than under present conditions. The chartered rights should, however, be carefully made complete and secure before any work is begun or the city definitely committed to large expenditure in this direction, so that ultimately, perhaps fifteen years hence, or perhaps not until thirty years hence, Greater New York can, if it needs, divert the entire flow in a dry year.

To the above $2,000,000 for all mill site and water-power damages, add for possible damages to various established industries, marble quarry and marble finishing plant, also for contingencies on land values, a sum which, although I have made it up as a sum of various items, is little better than a guess, since much depends on the promptness and tact with which these matters are negotiated and adjusted, with consequent saving of litigation—say $500,000

The above lands, dwellings and water rights are for a supply of 750 to 800 million gallons per day. To illustrate that the above takings of lands and houses are not in excessive proportion to what has been done elsewhere, we make the following comparison:

On the Massachusetts Metropolitan Works now under construction for a supply of.....	111 million gallons per day.
The total land taken is estimated at.....	5,163 acres.
This includes of margins and islands.....	968 acres.
The number of dwellings and buildings to be removed is about....	360.
A large proportion of these lands and dwellings have been already purchased by the Metropolitan Water Board through friendly agreement with owners, without recourse to litigation, at prices averaging, for equal conditions, probably less than herein assumed.	
The Metropolitan Reservoir required relocation of 6.56 miles of railway and overflowed 19.25 miles of highways, requiring new roads built.	
The Boston Metropolitan Works are for the additional supply of a present population of about one million people. The Housatonic system is proposed for a present population of three and a half millions.	
The Massachusetts Metropolitan Works require removal of a large cemetery containing.....	4,500 interments.
—or 50 per cent. more than the total above the Housatonic Dam.	
The total area of reservoir bottom to be stripped of organic matter is nearly double that of the south arm of the Ten Mile Basin herein proposed to be stripped.	

Removal of cemeteries above Housatonic Dam:

Previously estimated above Webatuck, 1,825 interments, cost...	$80,000
Additional, above Merwinsville, 1,045 interments, cost, say....	45,000
Probable cost of suitable grounds, landscape work, reinterment and removal and resetting of monuments.....	$125,000

In order to obtain a safe outside figure there has been included in the foregoing one large cemetery, with about 600 interments, which being 50 feet above flow line, need not be disturbed.

Removal of bushes, trees and stumps from entire surface to be flowed.

Previously estimated above Webatuck Dam, 860 acres, at $70	$60,000	
Below Webatuck Dam, 2.5 square miles, 1,600 acres, at $70	112,000	
	$172,000	$172,000

(Eight hundred and fifty acres grubbing on Massachusetts Metropolitan Reservoir was recently contracted for at $70 per acre.)

Removal of organic matter from sites of dwellings and farm yards, less value of old material. Removal of muck and mud from the few natural ponds and swamps in flowed area, or covering same with clean gravel.

Previously estimated for about 500 dwellings and 50 acres ponds above Webatuck Dam	$75,000	
For additional below Webatuck Dam, 500 dwellings and perhaps 300 to 400 acres ponds and swamps (including covering swamp between Hatch and Leonard Ponds with gravel)	425,000	
		500,000
Stripping all loam, turf, muck and vegetable matter from south-east arm of reservoir for 9.5 miles in length, 3,100 acres, at $700 per acre		2,170,000

(Recent awards of contracts for stripping about 2,000 acres on Massachusetts Metropolitan Reservoir average $610 per acre.)
None of New York's present reservoirs have had bottom stripped in this manner.

Change in location of Berkshire Division, N. Y., N. H. & H. R. R., for 20.4 miles from New Milford to W. Cornwall, Conn., per estimate of Horace Ropes, C. E. (see below for details)	$980,000	
Previously estimated for Harlem R. R.	1,000,000	
		$1,980,000

Rough Estimate of Cost of Relocation of a Portion of the Berkshire Division of N. Y., N. H. & H. R. R.

Between New Milford and West Cornwall		20.4 miles.
Sidings		2.6 "
Grading—		
20.4 miles of main line	$410,000	
2.6 " " sidings	26,000	
1,000 lineal feet tunnel—excavation, $57,000; masonry, $20,000	77,000	
		$513,000
Bridges and Culverts—		
3 iron trestles, 300 ft. x 50 ft., at $17,000	$51,000	
1 " " 500 ft. x 30 ft	26,000	
2 10-ft. arch culverts, 60 ft. between headwalls, at $3,600	7,200	
3 6-ft. arch culverts, 60 ft. between headwalls, at $2,200	6,600	
3 3 ft. x 3 ft. box culverts, 45 ft. between headwalls, at $600	1,800	
40 2 ft. x 2 ft. box culverts, 45 ft. between headwalls, at $400	16,000	
11 highway bridges for separation of grades, at $6,000	66,000	
		174,600
Track—		
20.4 miles main track, laid and ballasted, at $7,800	$159,120	
2.6 miles of side track, at $5,900	15,340	
		174,500
Station Buildings—		
3 combination passenger and freight depots		12,000
Telegraph Line—		
Rebuilding 20.4 miles, at $400		8,200
Fencing—		
30 miles of single fence, at $200		6,000
Right-of-Way—		
Additional outside of basin, 156 acres, at $150		23,400
		$911,700
Add 10 per cent. for engineering and contingencies		91,200
Total		$1,002,900
Less value of old rails, etc		31,700

Net cost about $971,200, or say, $980,000.

$$\frac{980{,}000}{20.4} = \text{about } \$48{,}500 \text{ per mile.}$$

New Highways.

The most satisfactory compensation for highways flooded would probably be rendered by building an equivalent length of good macadam road in each town on such lines as would best serve the public convenience rather than by merely building good marginal roads about the reservoir, which would be out of the line of regular travel. The total length of highways and crossroads flowed, plus the length on islands and peninsulas, made useless, is, in New York 42 miles, in Connecticut 48 miles, perhaps 25 per cent. of these are unimportant crossroads. The present roads are of an average class that could be built probably at a cost of from $2,000 to $5,000 per mile.

A study of the road maps and roads in vicinity of reservoir indicates that in addition to the 35 miles of substitute macadam roads estimated for the Ten Mile Reservoir with its dam above Webatuck, costing	$700,000	
There would probably be required for the Housatonic plan about 3 miles in New York and 47 miles in Connecticut. Some of the relatively unimportant roads could properly be ordinary gravel roads, and not macadamized, thus reducing the average cost, 50 miles, at $20,000 per mile	1,000,000	
		$1,700,000

An equal length of roads better than the present roads could probably be constructed for a fifth part of this sum, and the only reason for including such an enormous figure in the estimate is found in regarding this as a form of compensation to these towns for this invasion and the transformation of certain of their valleys into a beautiful lake.

Our estimate should be large enough and compensation to the towns is likely to be required in one form or another.

On the Massachusetts Metropolitan Works now under construction for 111 million gallons per day, 19.2 miles of highways are to be flooded.

Bridges at Cornwall and near North Kent, say	$100,000
Previously estimated for bridges near Hurd's, Wing and Wassaic	175,000

Channel in bottom of reservoir from Pawling up to Wing for purpose of drawing reservoir down to elevation 425, a portion of material excavated being deposited for a broad marginal lawn between Pawling Cemetery and Reservoir, as previously estimated	$200,000

Outlet Dam and Sluiceways at Pawling.

Dam of earth, with heavy masonry core wall to ledge, greatest height above natural surface, 40 feet; total length, 700 feet, as previously estimated	85,000
Sluiceways and gates, hoisting apparatus, gate-house and keeper's house	75,000
Storm water channel at Pawling and diversion of two brooks	50,000
Probable cost of strip of wet, unused swamp land along both sides of canal, averaging 1,600 feet wide for 60,000 feet in length, or 11.5 miles, 2,200 acres, as previously estimated	70,000
Open canal, 45 feet bottom width, 10 feet ordinary depth of water, side slopes 3 to 1, no walls, excavated through swamp south from Pawling outlet dam to head of East Branch Reservoir—11.5 miles—as previously estimated, plus allowance for increase in width; about 3,200,000 cubic yards, excavation, at 25 cents	800,000

From surface indications, excavation soft sand that can be taken out by a floating dredge for one-half of the above figure; or, indeed, a considerable portion in length could be omitted, except for desire to improve quality of water by draining swamp.

Sewage disposal works at Pawling and Patterson—say	100,000

SUMMARY OF ESTIMATES OF COST.

TABLE NO. 52.

Supply of 150 Million Gallons per 24 Hours from Ten Mile River; Also for a Supply of 750 Million Gallons per 24 Hours from Ten Mile and Housatonic Rivers.

Each delivered into East Branch (Sodom) Reservoir in the Croton Watershed at an elevation of 415 feet above sea level.

For details and unit prices, see previous pages.

	Ten Mile River.	Housatonic and Ten Mile Rivers.
	150 million gallons.	750 millon gallons.
Dam and waste way channels complete	$1,605,000	$5,965,000
Land, dwellings, and other buildings on reservoir site	2,307,000	4,674,000
Water rights, mills and mill sites and injury to established business and legal expenses	900,000	2,500,000
Removal of certain cemeteries to new locations	80,000	125,000
Grubbing and removal of trees, stumps and bushes	60,000	172,000
Removal of mud and muck from ponds and cleaning up sites of dwellings	75,000	500,000
Stripping bottom of southwest arm of reservoir, 3,000 acres.	2,170,000	2,170,000
Change in location of Harlem and Berkshire railroads	1,000,000	1,980,000
New highways (and compensation to towns)	700,000	1,700,000
Bridges	175,000	275,000
Outlet dam at Pawling, with gates and sluices	135,000	160,000
Deep channel Pawling to Hurd's and above	200,000	200,000
Storm water channel and filter dam at Pawling and diverting brooks at Pawling	50,000	50,000
Sewage disposal works for Pawling and Patterson	100,000	100,000
Deep open channel for draining swamp and improving East Branch Croton, including cost of 2,200 acres swamp land between Pawling and Milltown	620,000	870,000
For Ten Mile only	$10,177,000	.....
For Ten Mile and Upper Housatonic	.....	$21,441,000
Add 15 per cent. for contingencies and supervision	1,526,000	3,216,000
Add for contingencies on real estate and other settlements	1,000,000	2,000,000
Estimated total cost of water, water rights, lands, buildings, damages, legal expenses and cost of all structures necessary for collection, storage and delivery of entire safe yield of water into the present East Branch Reservoir in the Croton Watershed	$12,703,000	$26,657,000

TRANSPORTATION OF TEN MILE AND HOUSATONIC WATER TO NEW YORK.

Aqueduct Capacity Required.

There is already some surplus Croton Aqueduct capacity which could, if needed, be used temporarily for conveying a part of this additional water supply coming from the Ten Mile river, but this surplus aqueduct capacity is small.

The New Aqueduct, if kept clean, will convey	300	mil. gals. daily.
The Old Aqueduct, if kept clean, will convey	80	"
Total Croton Aqueduct capacity	380	"
The safe yield of the Croton Watershed, permanently, is	275	"
Leaving an apparent surplus aqueduct capacity of	105	"
Not all of this can be availed of at all times; a moderate degree of foulness may lessen it 10 per cent	38	"
The demand for water averages greater in February and September by from 2 per cent. to 5 per cent., or say	17	"
Leaving as the real surplus of aqueduct capacity, about	50	"

In planning new aqueducts for the additional supply it is safer to disregard this and, indeed, will in the long run prove cheapest to make the new aqueduct large enough to also convey the 50 million gallons of water that the East Branch Division of the Croton Watershed can yield, and so bring this into the city at an elevation of 300 feet above sea level, instead of allowing this extra head to be wasted, as now.

Margin for Slime and Extra Draft.

In the new aqueducts, in recognition of the fact that any future aqueduct is liable to suffer the same impairment of carrying capacity now found in the New Croton Aqueduct, 15 per cent. will be added as a margin of safety for delivery in case the aqueduct walls remain unswept for a long period. A further allowance of 10 per cent. will be added to provide for the draft in September and February, which is commonly larger than the mean draft, and also as a means for promptly refilling a reservoir after the aqueduct has been shut off for repairs.

Therefore, the aqueduct considered for the Ten Mile, apart from the Upper Housatonic, will be designed of size proper to convey, when clean, 150 + 50 = 200 + 25 per cent = ... 250 million gallons per day.

and the aqueduct for the Ten Mile and Housatonic combined,

$750 + 50 = 800 + 25$ per cent. =..... 1,000 million gallons per day.

Aqueduct Possibly Built in Two Instalments.

An estimate will also be presented for an aqueduct of just half the last-mentioned size, or appropriate for ultimately providing two aqueducts, instead of one; although from such study as I have found time to give this matter, I am of the opinion that if the increase in use and waste of water is to continue at the same rate as in the past it would prove most economical and entirely safe to construct a single aqueduct line for the combined Ten Mile, Housatonic and East Branch water, making the tunnels and all masonry structures full size from the start and deferring only the steel pipe siphons and similar structures that can be as cheaply added at any future time. If it were fairly certain that thorough and efficient waste prevention would be maintained, possibly a different conclusion would be reached. The carrying capacity of an aqueduct increases very much faster than the cost, as size is increased. This will be seen from Diagram No. 66 at the left-hand end. *The best and cheapest method of safeguard against interruption of flow is to provide ample storage reservoirs near the city, like Jerome Park, Central Park, Park Hill and the enlarged Rye Pond and future Kensico and perhaps Pocantico.*

Possible Lines for Aqueduct from Sodom to Rye.

For determining approximately the best and cheapest line for an aqueduct from the East Branch Reservoir to New York, a reconnaissance was made by Mr. Horace Ropes, who, as previously stated, had had much previous experience in the construction of railroads, tunnels and aqueducts and in field reconnaissance. Under my instructions, he explored certain lines with two very different characteristics; namely,

First—A masonry aqueduct line chiefly in "cut-and-cover," following the surface of the ground and thus capable of being quickly reached and repaired in case of any great disaster. With a steel pipe line or a cut-and-cover aqueduct, an accident that could shut off the supply for more than a week, before the break or stoppage could be repaired with the large number of workmen that could quickly be brought to bear upon it, is almost inconceivable. From this point of view, the aqueduct has an advantage in safety over the deep tunnel, in addition to its great advantage in smaller cost per mile. In modern masonry aqueducts, like the Sudbury or the Wachusett, such care is given to the foundation, and so ample a factor of safety used in the masonry, that the safety, solidity and permanence are fully comparable with that of a deep tunnel.

Second—A location was sought chiefly in deep tunnel with good cover, and with shafts of reasonable depth.

Third—A line from East Branch (Sodom) Reservoir to Rye Pond was also explored, about half in cut-and-cover, and half in tunnel, in the hope of finding the cheapest combination of which the ground would admit. Time and means did not admit of instrumental surveys, so the possible routes were first studied in the office upon the admirable topographical maps of the United States Geological Survey and then explored in the field by Mr. Ropes, with the aid of pocket level and aneroid barometer, making such departure from the lines first laid down as the characteristics of the ground required.

Rye Pond Reservoir.

A brief study was also made by Mr. Ropes, under instructions to investigate the feasibility of raising the level of Kensico Reservoir to about 345 feet above sea level and thus obtain a large equalizing reservoir intermediate between the beginning and the end of the aqueduct; or in case this should prove too formidable, to seek to raise the level of Rye Pond only, by a dam between that and Kensico. The latter course appeared the better and is followed in the estimates; it is much cheaper and will serve for many years, while Kensico can be raised at some time in the distant future if additional storage is needed half way to New York.

Brief reconnaissance was also made of a possible alternative site for a great equalizing reservoir in the Pocantico Valley, between Merritt's Corners and a point 1¼ miles below Whitson, considering this as a substitute for Rye or the enlarged Kensico. This was found to be apparently feasible, but although of large capacity does not appear so attractive as the plan of raising Rye Pond by a dam between Rye and Kensico. It merits full survey in the final studies.

Characteristics of the Ground Between Sodom and Rye, for Aqueduct Building.

The district traversed was generally a very rough country, consisting of a succession of prominent hills and ridges with seldom more than ravine-like valleys between. The hills were scantily covered with soil, except on the northerly and westerly slopes, where drift deposits were common, usually on the lower portions of the slopes. Near the summits and on the other slopes frequent ledge outcrops left no doubt as to the general presence of ledge at shallow depths.

Back from the railroad, this region is still a farming district. Not more than three or four places were seen which exhibited the pretensions of "a gentleman's estate."

The tillable lands do not look very fertile, much of the cleared land is simply pasture, and a large proportion of the hillsides and summits are woodland. These back farms were said to be worth from $40 to $60 per acre.

Along the valley of the Croton river and of that branch of Cross river which heads near Chappaqua, the land in the bottoms and on the lower slopes is better, but higher up where the west line is located, it is generally of an inferior character. These farms also seem to be almost wholly in the hands of actual farmers.

At the Towns of Katonah, Mt. Kisco, Chappaqua and Pleasantville there are signs of growth and improvements which betoken the arrival of "city people," with a consequent stiffening in values for farm lands and boom prices for town lots.

The westerly line explored for the Aqueduct will cross some town property and through or very near some dwellings in Mt. Kisco, Chappaqua and Pleasantville. Right-of-way at these places will be found probably the most expensive of any along the line, yet not sufficiently so to have any material effect in the total cost.

Geologically, the prevailing formation of the district seemed to be granite, much of it apparently of a gneissoid or schistose character; true schists were noted at some places, and other varieties of rock showed in limited areas. The outcrops of ledges showed variations from hard and massive, to very seamy and broken and sometimes "rotten"; the greater

part, however, was found to be firm rock, apparently well adapted for economical tunneling.

On the profiles mentioned above special note was made of the characteristics of the work required on different sections. A part of these profiles are reproduced in outline and on a small scale in Figures 86 to 90 for the purpose of making the relation of the several types of conduit more clear.

East Line—Sodom to Rye.

The cut-and-cover portions of the East Line closely follow some of the smaller valleys, with the result that, say one-half of the grading will be fairly easy. At the head of the valley the tendency will be for the line to run into continuous deep cutting of considerable length, and at the lower end of the valley, to be above the bottoms and hugging the hill slopes. On these slopes, and also in the deeper cuttings in the bottom of the valley, a large part of the excavation may be ledge. Through these valleys an abundance of sand and some gravel is to be had for masonry. No ledges were observed which gave promise of stone for fine masonry, though it is not unlikely some may be found near the line.

The tunnels will all be in rock. For more than one-half their length the indications are that this rock will be firm and durable, but the rock in some localities showed evidence of being much shaken and blocky, tending to make the work difficult and dangerous. Considerable water may also be expected at these places. It is believed, however, that, like the Wachusett Tunnel, not more than one-half the length of these tunnels will require arching, and that amount has been allowed for in the following estimates of cost.

West Line—Sodom to Rye.

The location of the West Line is such as to take it across some of the main drainage of the country at such a high level as to preclude the possibility of embankments or bridges, and pipe siphons are indicated for those places. At two or three of the smaller dips (40′–60′ below grade), embankments or bridges may be deemed best.

The cut-and-cover portion of the line closely follows the trend of quite steep slopes, being in many places 60 feet or more above the bottom of same. At points where the line is highest the slopes are usually rugged, and heavy rock excavation will be encountered; at lower levels the slopes are more regular, and some benefit will be derived from the drift deposits. It seems possible that a large share (say one-half) of the line can be located so that the grading will range from light to moderately heavy work.

The tunnels are all so short that no shafts will be needed. They will be through ledge, and only one-half of their length is estimated as lined with masonry.

Even though the first aqueduct were designed to carry only a fraction of the supply, it might be better to make all the tunnels of full size at the start.

The branch line to the suggested alternative Pocantico Reservoir, shown on the map, was not examined on the ground because of pressure of other parts of the work. From general knowledge of the country, however, Mr. Ropes predicts it will be quite rough, with much rock work.

Siphons.

These are designed to be of steel pipe, following the specifications of page 321. The number of lines of pipe in each and their diameters are given in the tabular estimate. It is obvious that the laying of more than the first string of pipe at each of these siphons can be deferred, and thus effect

some saving. The cost of the future siphons has, therefore, been discounted to present value at 3 per cent. interest.

Special Structures.

All the usual accessories of such an aqueduct, as minor culverts, blow-offs, man-holes, etc., will have to be provided in about the usual quantity. Of those which may be termed special structures, there will be on the East Line, a gate-house at East Branch Reservoir, masonry bridges at Titicus and Cross rivers, and a terminal structure at Kensico Reservoir.

On the West Line, in addition to the gate-house and the terminal structure at the reservoirs, there will be masonry bridges for the pipe line across the Titicus, Cross and Stone Hill rivers, and large culverts over one stream on mile 10, and two streams on mile 19. Eight siphon chambers will be required.

On miles 3 and 9 of the West Line, the aqueduct crosses valleys which are 60 feet below aqueduct grade, and perhaps a little less than 1,000 feet wide at the grade line. Without much doubt the cheapest plan would be to use pipe siphons at these places, but since the stretches are short, and it is desirable not to break the continuity of the masonry aqueduct oftener than is necessary, these places are treated as high embankments.

Three plans were, however, briefly considered for the crossing on mile 3, West Line, namely:

1st. An embankment with a 12-foot arch culvert for the brook, and two 20-foot arches for the highways.

2d. A pipe siphon, with two siphon chambers and a 12-foot culvert for the brook.

3d. A masonry bridge 650 feet to 700 feet long, somewhat as suggested on profile.

Rough computations indicate the comparative cost to be for embankment about $114,000 for either the small or the large aqueduct; for siphon, $75,000 for the small, and $118,000 for the larger capacity; and for the bridge, probably not less than $200,000 for the small aqueduct.

Right-of-Way.

Right-of-way is assumed to average 50 feet wide over the tunnels, 100 feet wide for the smaller aqueduct in trench, and 150 feet wide for the larger. This has been estimated to cost the uniform price of $200 per acre, with the following additions for special cases. Where tunnels are built, there is strong

probability of draining wells and springs near the line, and an allowance of $1,500 per mile has been made for this item.

Also, additional ground must be acquired for waste banks at the portals and shafts, say 3 acres at each point. The extra damages at Salem Centre and Cross River Village have been estimated at $5,000 each. At Mount Kisco, Chappaqua, and Pleasantville, at $20,000 each.

Surveys by New York Department of Water Supply.

Mr. Ropes further reports:

"While going over the line and near the end of my reconnaissance, I learned that Mr. G. W. Birdsall, Chief Engineer, is investigating the problem of drawing 175 million gallons from the Ten Mile river, and has for some time had several parties in the field making surveys.

"I learned from a very reliable source that a survey had been made for a combination tunnel and cut-and-cover aqueduct on about the same location as my east line, but there seemed to be an inclination to reject this for a line which should be wholly in tunnel. The surveys for this latter were in progress at that time.

"Surveys had also been completed of the Kensico and Rye Pond Reservoirs to determine the possibility and cost of raising the water level to elevation 355. This work included plans and estimates for a large dam at the outlet of Kensico Reservoir, and a smaller one just above Kensico Village. A pipe line had been surveyed from Kensico Reservoir to as far south, I think, as the Harlem river, also an alternate line which should be mainly or wholly in tunnel. In this case, also, a preference was expressed for the tunnel construction."

TABLE NO. 53.

HOUSATONIC AND TEN MILE RIVERS PROJECTS.

Estimated Cost of Conduits EAST BRANCH RESERVOIR *to* RYE RESERVOIR *via* EAST LINE, *Distance 22 Miles, About 50 per Cent. in Tunnel.*

Assumptions Hyd. Grade Elev., 385 and 350.		Capacity 250 Million Gallons, Average Diameter 10.6 Feet.		Capacity 500 Million Gallons, Average Diameter 13.5 Feet.		Capacity 1,000 Million Gallons, Average Diameter 17.7 Feet.	
22 miles C. & C. and Tunnel at 1.55 ft. per mile ..	34.1						
Minor losses of head	0.9						
Total head expended	35.0	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.
10.2 miles cut and cover aqueduct		$142,000	$1,448,000	$189,000	$1,927,800	$265,000	$2,703,000
5.9 miles arch lined tunnel		322,000	1,899,800	428,000	2,525,200	603,000	3,557,700
5.9 miles side lined tunnel		260,000	1,534,000	350,000	2,065,000	485,000	2,861,500
1,230 lineal feet shaft excavation		100	123,000	125	153,800	150	184,500
Gate House at E. Branch Reservoir		.....	30,000	.....	39,000	.....	50,000
Terminal structure at Rye Reservoir		.....	10,000	.....	13,000	.....	17,000
Bridge and embankment at Titicus River		.....	92,400	.....	92,800	.....	93,400
Bridge and embankment at Cross River		.....	39,800	.....	40,000	.....	40,200
Culverts, manholes, etc		.....	42,500	.....	43,300	.....	44,400
Rye Dam and Reservoir complete. Proposed flow line 355		.....	500,000	.....	500,000	.....	500,000
10.2 miles cut and cover right of way		.....	34,500	.....	40,600	.....	48,800
11.8 miles tunnel right of way		.....	44,500	.....	44,500	.....	44,500
			$5,798,500		$7,485,000		$10,145,000
Add 15 per cent. for contingencies and supervision, etc.		.....	869,800	.....	1,122,800	.....	1,521,800
Total		.....	$6,668,300	.....	$8,607,800	.....	$11,666,800

TABLE NO. 54.

HOUSATONIC AND TEN MILE RIVERS PROJECTS.

Estimated Cost of Conduits, EAST BRANCH RESERVOIR *to* RYE RESERVOIR, *via* DIRECT LINE, *Almost Wholly in Tunnel.*

Assumptions Hyd. Grade Elev., 385 and 340. 18.8 miles tunnel at 2.0′ per mile 37.6; 1.2 miles siphon at 5.0′ per mile 6.0; 1.0 canal at ends 0.4; Minor losses of head 1.0; Total head expended 45.0	Capacity 250 Million Gallons. Average Diameter inside Arched Lined Tunnel, 11 feet.		Capacity 500 Million Gallons. Average Diameter A. L. T. 14 feet.		Capacity 1,000 Million Gallons. Average Diameter A. L. T. 18.4 feet.	
	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.
1.0 mile open channel	.....	$160,300	.....	$228,300	.....	$315,700
9.4 miles arch lined tunnel	$307,000	2,885,800	$402,000	3,778,800	$563,000	5,292,200
9.4 miles side lined tunnel	248,000	2,331,200	330,000	3,102,000	456,000	4,286,400
*1.2 miles steel pipe for siphon	7′ 1″ diam. 122,000	292,800	7′ 11″ diam. 144,000	345,600	8′ 6″ diam. 162,500	390,000
Present value of additional pipes:						
2d line of pipe deferred	.....	.....	18 years	101,500	21 years	209,700
3d line of pipe deferred	.....	.....	.....	.....	42 years	56,300
1,070 lineal feet shaft excavation	100	107,000	125	133,800	150	160,000
Gate House at E. Branch Res.	.....	30,000	.....	39,000	.....	50,000
Terminal structure at Rye Res.	.....	10,000	.....	13,000	.....	17,000
Bridges and embankments at Titicus, Cross and Stone Hill rivers	.....	77,900	.....	89,000	.....	109,200
6 siphon chambers	10,000	60,000	12,600	75,600	16,000	96,000
Rye Dam and Res. complete, proposed flow line 355	.....	500,000	.....	500,000	.....	500,000
19.5 miles right of way 50 feet wide and 3 acres extra at shaft, etc.	.....	61,100	.....	61,100	.....	61,100
		$6,516,100		$8,467,700		$11,543,600
Add 15 per cent. for contingencies and supervision, etc.	.....	977,400	.....	1,270,200	.....	1,731,500
Total	.....	$7,493,500	.....	$9,737,900	.....	$13,275,100

* NOTE—Assumptions for lines of pipe required are as follows:
250 million gallons, two lines 7′ 1″ diameter pipes, both laid at beginning.
500 million gallons, three lines 7′ 11″ diameter pipes, 2 laid at first and third 18 years later.
1,000 million gallons, five lines 8′ 6″ diameter pipes, 2 laid at first, third and fourth 21 years later, and fifth 42 years later.
Capacity of pipe lines when foul to be 220, 410 and 880 mil. gals.

TABLE NO. 55.

HOUSATONIC AND TEN MILE RIVERS PROJECTS.

Estimated Cost of Conduits. EAST BRANCH RESERVOIR *to* RYE RESERVOIR. *Via* WEST LINE, *Distance* 26.4 *Miles, Principally Cut and Cover Aqueduct.*

Assumptions Hyd. Grade Elev. 385, 353.		Capacity 250 Million Gallons. Average Diameter C. & C. 11.7 feet.		Capacity 500 Million Gallons. Average Diameter C. & C. 15.4 feet.		Capacity 1,000 Million Gallons. Average Diameter C. & C. 20.2 feet.	
3.35 miles siphon at 3.5'	11.73						
23.05 miles C. and C. tunnel at 0.8'	18.44						
side-lined tunnel, 1.0'	.23						
Minor losses of head	1.60						
Total	32.00	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.
20.45 miles cut and cover aqueduct		$160,000	$3,272,000	$221,000	$4,519,500	$314,000	$6,421,300
1.45 miles arch-lined tunnel		364,000	527,800	501,000	726,500	722,000	1,046,900
1.15 miles side-lined tunnel		1 foot per mile 284,000	326,600	1 foot per mile 390,000	448,500	1 foot per mile 538,000	618,700
*3.35 miles steel pipe for siphons		7 ft. 7 in. diam. 136,000	911,200	8 ft. 6 in. diam. 162,000	1,085,400	8 ft. 6 in. diam 162,000	1,085,400
Present value of additional pipe:							
2d line of steel pipe deferred				18 years.	318,800	18 years.	637,600
3d line of steel pipe deferred						36 years.	374,500
Gate House at E. Branch Res.			30,000		39,000		50,000
Terminal structure at Rye Res.			10,000		13,000		17,000
Bridges and embankments on mile 3			111,500		112,200		113,000
Bridges and embankments on mile 9			74,800		75,300		76,000
Bridges at Titicus, Cross and Stone Hill rivers, also at miles 10 and 19 for siphon crossings			132,000		149,200		196,100
8 siphon chambers		10,000	80,000	$12,600	100,800	$16,000	128,000
Culverts, manholes, etc			61,500		64,600		68,600
Rye Dam and Res. complete, proposed flow line 355			500,000		500,000		500,000
20 miles C. & C. right of way			108,000		120,200		136,000
3.35 miles siphon right of way			8,000		12,700		18,700
2.30 miles tunnel right of way			11,100		11,100		11,100
			$6,164,500		$8,296,800		$11,498,900
Add 15 per cent. for contingencies and supervision, etc.			924,700		1,244,500		1,724,800
Total			$7,089,200		$9,541,300		$13,223,700

*NOTE—Assumptions for lines of pipe required are as follows:
250 million gallons—two lines 7 feet 7 inches diameter pipes, both laid at beginning.
500 million gallons—three lines 8 feet 6 inches diameter pipes, two laid at first and third 18 years later.
1,000 million gallons—six lines 8 feet 6 inches diameter pipes, two laid at first, third and fourth 18 years later, and fifth and sixth 36 years later.
Capacity of pipe lines when foul to be 220, 440 and 880 mil. gals.

TABLE NO. 56.

HOUSATONIC AND TEN MILE RIVERS PROJECTS.

Estimated Cost of Conduits, RYE RESERVOIR *to* CITY LIMITS (*Woodlawn*) *via* HIGH LINE—*Distance Rye Reservoir to Park Hill* 13.8 *miles* (55 *per cent. Cut and Cover and* 45 *per cent. Pipe Line*), *Plus* 0.6 *Miles to City Limits.*

Assumptions Hyd. Grade Elev. 326 and 303.		Capacity 250 Million Gallons. Average Diameter C. and C. 11.7 feet.		Capacity 500 Million Gallons. Average Diameter C. and C. 15.4 feet.		Capacity 1,000 Million Gallons. Average Diameter C. and C. 20.2 feet.	
7.6 miles C. and C. aq. at 0.8′ per mile	6.08						
6.2 miles steel pipe 2.5′ per mile	15.50						
Minor losses of head	1.42						
Total head	23.00	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.
7.6 miles cut and cover aqueduct		$160,000	$1,216,000	$221,000	$1,679,600	$314,000	$2,386,400
*6.8 miles steel pipe line		8′ 2″ diam. 151,000	1,026,800	8′ 2″ diam. 151,000	1,026,800	8′ 6″ diam. 163,500	1,111,800
Present value of additional pipes:							
2d line steel pipe deferred		6½ years	847,300	6½ years	847,300	7⅛ years	890,700
3d line steel pipe deferred				13 "	699,200	15 "	713,600
4th line steel pipe deferred				20 "	568,500	23 "	563,300
5th line steel pipe deferred						30.5 "	451,300
6th line steel pipe deferred						38 "	361,600
7th line steel pipe deferred						46 "	285,500
3 siphon chambers		$13,300	39 900	$16,800	50,400	$21,300	63,900
Bridges on miles 1 and 10			78,200		99,900		130,800
Culverts, manholes, etc		Per mile, 2,000	27,600	2,500	34,500	3,200	54,200
2.5 miles right of way		" 6,000	15,000	6,000	15,000	7,200	18,000
11.3 miles right of way		" 3,000	33,900	3,000	33,900	3,800	42,900
Park Hill Reservoir, North Basin			3,696,000		3,696,000		3,696,000
0.7 miles by-pass aqueduct		163,000	114,100	210,000	147,000	297,000	207,900
			$7,094,800		$8,898,100		$10,977,900
Add 15 per cent. for contingencies and supervision, etc.			1,064,200		1,334,700		1,646,700
Total			$8,159,000		$10,232,800		$12,624,600

*NOTE—Assumptions for lines of pipe required are as follows:
250 million gallons, two lines 8′ 2″ diameter pipes, one laid at first, and second 6½ years later.
500 million gallons, four lines 8′ 2″ diameter pipes, one laid at first, second 6½ years later, third 13 years later, and fourth 20 years later.
1,000 million gallons, seven lines 8′ 6″ diameter pipes, two laid at first, second 7⅛ years later, third 15, fourth 23, fifth 30½, sixth 38, and seventh 46 years later.
Capacity of pipe lines when foul to be 220, 440 and 880 mil. gals.

AQUEDUCT FROM RYE POND TO PARK HILL.

A further brief study was made of various possible lines between Rye Pond and the proposed high-pressure terminal reservoir at Park Hill near the city limits; comparative estimates being made for a steel pipe, a deep tunnel, and for a line containing the largest possible proportion of cut-and-cover masonry aqueduct. For these studies, profiles were made up from the geological survey maps and the aneroid corrections. In all important aneroid work a recording stationary barometer was used for determining the atmospheric correction from hour to hour. The time for field work was necessarily brief, but is believed that the field work was sufficient to demonstrate the feasibility of each line and to serve in a rough and ready way for an estimate of its cost. Accurate traverse surveys, with levels and contours, borings and test-pits, would, of course, be needed before the superiority of one of these lines over the other could be demonstrated, but the natural tendency of more complete surveys would be toward finding a cheaper line, and with these three different lines explored and agreeing fairly well in cost, the estimate that is presented cannot be far from correct. It will be noted that the slopes and elevations of the several portions of the aqueduct were so carefully fixed as to make available nearly the whole of the storage capacity of the several storage reservoirs and to permit a wide range of heights, while securing substantially full delivery of the aqueducts.

Three different forms of conduit for this line were investigated and each found to be practicable; each has certain advantages.

First—A line having the largest possible proportion of cut-and-cover masonry aqueduct. This has the advantage in point of cheapness and is also favorable for prompt repair. Good supporting ground for cut-and-cover aqueduct, on the hydraulic grade, is found for more than half the entire distance.

Second—A line in deep tunnel, similar to the New Croton Aqueduct. This has the advantage in directness and possibly some advantage in operation under a head for permitting the reservoir to be drawn low.

Third—Steel pipe conduits. These have the advantage of permitting a part of the cost to be deferred, since one line of pipe only need be put in at the beginning, and the other lines added successively as the consumption increases.

Profiles and maps based on the topographical sheets of the U. S. Geological Survey, scale about one mile per inch, were prepared in the office from the best data available and taken into the field for adjustment to the character of the ground along the several lines. A careful preliminary study was made of the operation of the conduits and reservoirs with the reservoir levels at different heights, in the effort to so arrange the grades that it should be possible to maintain full delivery of the aqueduct while the reservoirs at either end were nearly full or nearly empty; thereby making the greatest possible amount of the storage volume available. It was found possible to accomplish this in a large degree with each of the three lines of aqueduct above mentioned, although it was, of course, found easier with the steel pipe and the tunnel under pressure, than with the line partly in cut-and-cover.

Estimates of the cost by each of these different lines will be found in the following tables.

TABLE NO. 57.

HOUSATONIC AND TEN MILE RIVERS PROJECTS.

Estimated Cost of Conduits, RYE RESERVOIR *to* CITY LIMITS (*Woodlawn*), *via* LOW LINE. *Distance, Rye Reservoir to Park Hill Reservoir*, 13.8 *miles* (*all* STEEL PIPES), *plus* 0.6 *miles to City Limits.*

Assumptions Hyd. Grade Elev., 340 and 303. 13.8 miles pipe line at 2.61′ per mile ..... 36.02′ Minor losses of head ..... 0.98′ Total head ..... 37.00′	Capacity, 250 Million Gallons. Average diameter S. Pipe, 8 ft. 1 in.		Capacity, 500 Million Gallons. Average diameter S. Pipe, 8 ft. 1 in.		Capacity, 1,000 Millon Gallons. Average diameter S. Pipe, 8 ft. 6 in.	
	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.
*14.4 miles steel pipe	{8′ 1″ diameter $148,500}	$2,138,400	{8′ 1″ diameter $148,500}	$2,138,400	{8′ 6″ diameter $162,000}	$2,332,800
Present value of additional pipes :						
2d line of steel pipe deferred	6½ years	1,725,000	6½ years	1,725,000	7½ years	1,869,000
3d line of steel pipe deferred	.....	.....	13 "	1,423,000	15 "	1,497,000
4th line of steel pipe deferred	.....	.....	20	1,157,000	23 "	1,182,000
5th line of steel pipe deferred	.....	.....	.....	.....	30.5 "	947,000
6th line of steel pipe deferred	.....	.....	.....	.....	38 "	758,700
7th line of steel pipe deferred	.....	.....	.....	.....	46 "	598,900
Bridge at Bronx river	.....	18,200	.....	21,900	.....	26,700
Bridge at Sprain brook	.....	15,200	.....	18,500	.....	22,800
Culverts, manholes, etc., 13.8 miles	2,000	27,600	2,500	34,500	3,200	44,200
2.5 miles, right of way	7,200	18,000	7,200	18,000	8,640	21,600
11.3 miles, right of way	3,600	40,700	3,600	40,700	4,320	48,800
Park Hill Reservoir, North Basin	.....	3,696,000	.....	3,696,000	.....	3,696,000
0.7 mile by-pass Aqueduct	163,000	114,100	210,000	147,000	297,000	207,900
		$7,793,200		$10,420,000		$13,253,400
Add 15 per cent. for contingencies and supervision, etc	.....	1,169,000	.....	1,563,000	.....	1,988,000
Total	.....	$8,962,200	.....	$11,983,000	.....	$15,241,400

*Assumptions for lines of pipe required are as follows :
250 million gallons, two lines, 8 feet 1 inch diameter pipes ; one pipe laid at first, and second 6½ years later.
500 million gallons, four lines, 8 feet 1 inch diameter pipes ; one pipe laid at first, and second 6½ years later ; third, 13 years later ; fourth, 20 years later.
1,000 millon gallons, seven lines, 8 feet 6 inches diameter pipes ; one pipe laid at first, and second, 7½ years later ; third, 15 years ; fourth, 23 years ; fifth, 30.5 years ; sixth, 38 years ; and seventh, 46 years later.
Capacity of pipe lines when foul to be 220, 440 and 880 mil. gals.

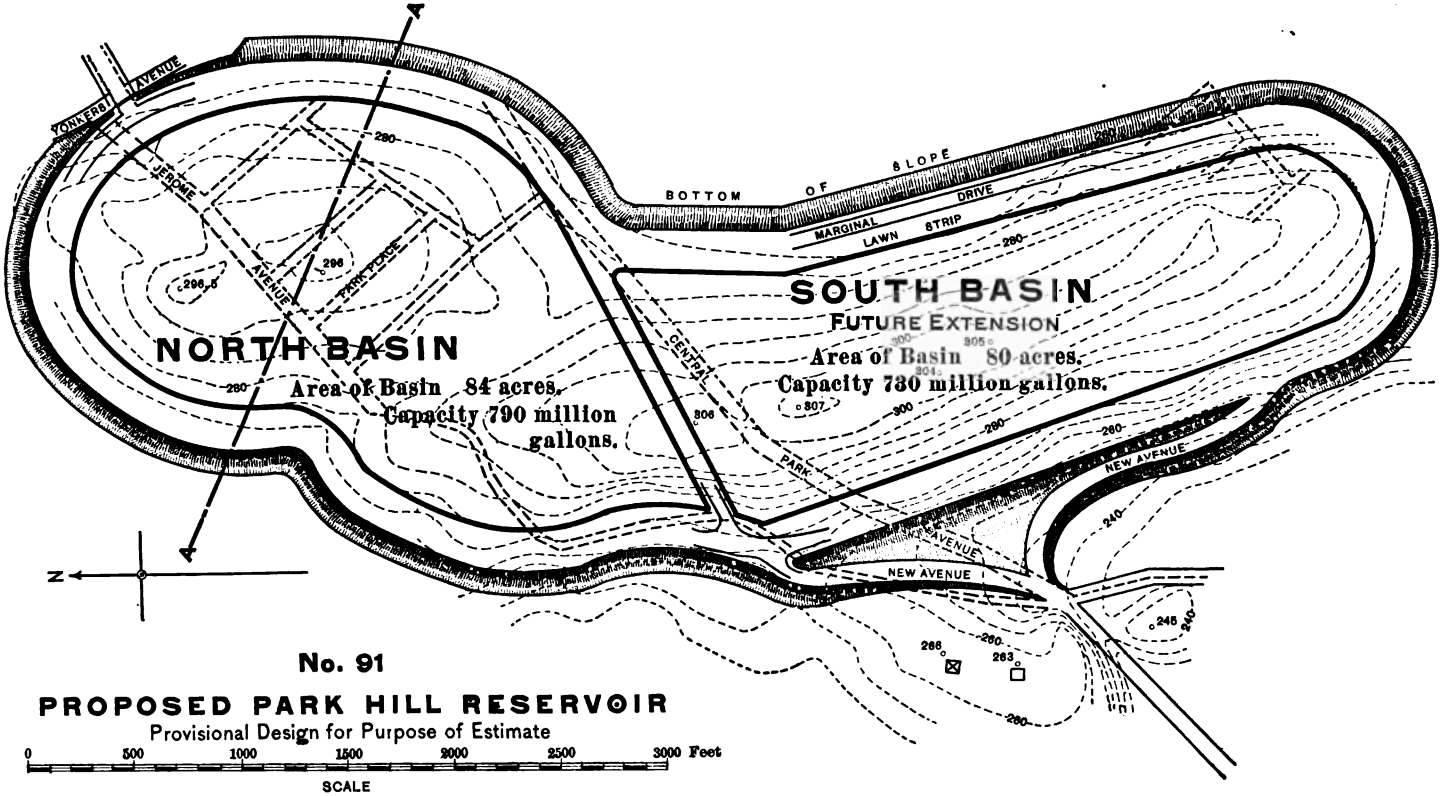
TABLE NO. 58.

HOUSATONIC AND TEN MILE RIVERS PROJECTS.

Estimated Cost of Conduits, RYE RESERVOIR *to* CITY LIMITS (*Woodlawn*). *Distance, Rye Reservoir to Park Hill Reservoir*, 12.4 *Miles* (*mostly* TUNNEL), *plus* 0.6 *Miles to City Limits.*

Assumptions Hyd. Grade Elev., 340 and 303. 11.5 miles tunnel at 2.74′ per mile ..... 31.5′ 0.9 miles steel pipe, 5.0′ per mile ..... 4.5′ Minor losses of head. ..... 1.0′ Total head ..... 37.0′	Capacity, 250 Million Gallons. Average Diameter A. L. T. 10.5 feet.		Capacity, 500 Million Gallons. Average Diameter A. L. T. 13.3 feet.		Capacity, 1,000 Million Gallons. Average Diameter A. L. T. 16.7 feet.	
	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.
11.5 miles arch lined tunnel	$292,000	$3,358,000	$378,000	$4,347,000	$522,000	$6,003,000
* 1.5 miles steel pipe line.....	7′ 1″ diam. 122,000	366,000	7′ 11″ diam. 144,000	432,000	8′ 6″ diam. 161,500	484,500
Present value of additional pipes :						
2d line of steel pipes deferred	.....	....	18 years	126,900	21 years	260,500
3d line of steel pipes deferred	.....	.....	.....	.....	42 years	70,000
1,120 linear feet of shaft excavation	100	112,000	125	140,000	150	168,000
4 siphon chambers	10,000	40,000	12,600	50,400	16,000	64,000
Gate House at Rye Reservoir.....	.....	30,000	.....	39,000	.....	50,000
Change of railroad crossing.....	.....	10,000	.....	10,000	.....	10,000
1 mile steel lining at low places (¾ cost of pipe).	135,000	135,000	181,000	181,000	238,200	238,200
Bridge at Bronx river.....	.....	18,200	.....	20,200	.....	24,200
Bridge at Sprain brook.....	.....	15,200	.....	17,000	.....	20,600
Right of way— Tunnel and shafts.	.....	49,800	.....	49,800	.....	52,000
Right of way— Additional at White Plains.....	.....	5,000	.....	5,000	.....	5,000
Park Hill Reservoir, North Basin.....	.....	3,696,000	.....	3,696,000	.....	3,696,000
0.7 mile by-pass Aqueduct	163,000	114,100	210,000	147,000	297,000	207,900
		$7,949,300		$9,261,300		$11,353,900
Add 15 per cent. for contingencies and supervision, etc.	.....	1,192,400	.....	1,389,200	.....	1,703,100
Total.....	.....	$9,141,700	.....	$10,650,500	.....	$13,057,000

Assumptions for lines of pipe required, as follows :
250 million gallons, two lines, 7 feet 1 inch diameter pipes, both laid at beginning.
500 million gallons, three lines, 7 feet 11 inches diameter pipes, 2 lines laid at first, and third 18 years later.
1,000 million gallons, five lines, 8 feet 6 inches diameter pipes, 2 lines laid at first, third and fourth 21 years later, and fifth 42 years later.
Capacity of pipe lines when foul to be 220, 440 and 880 mil. gals.



No. 91

PROPOSED PARK HILL RESERVOIR

Provisional Design for Purpose of Estimate

0 500 1000 1500 2000 2500 3000 Feet

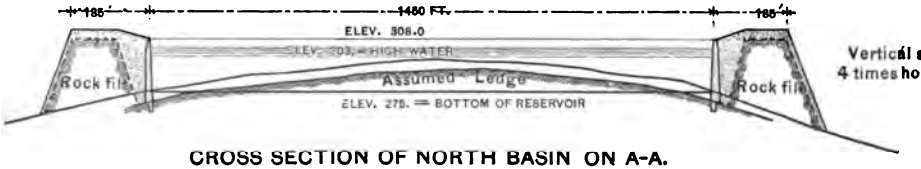
SCALE

Contours give Elevation above Sea Level. From Bench Mark of Croton Aqueduct Commission.

Surveyed April, 1900, by Horace Ropes and George A. Taber.

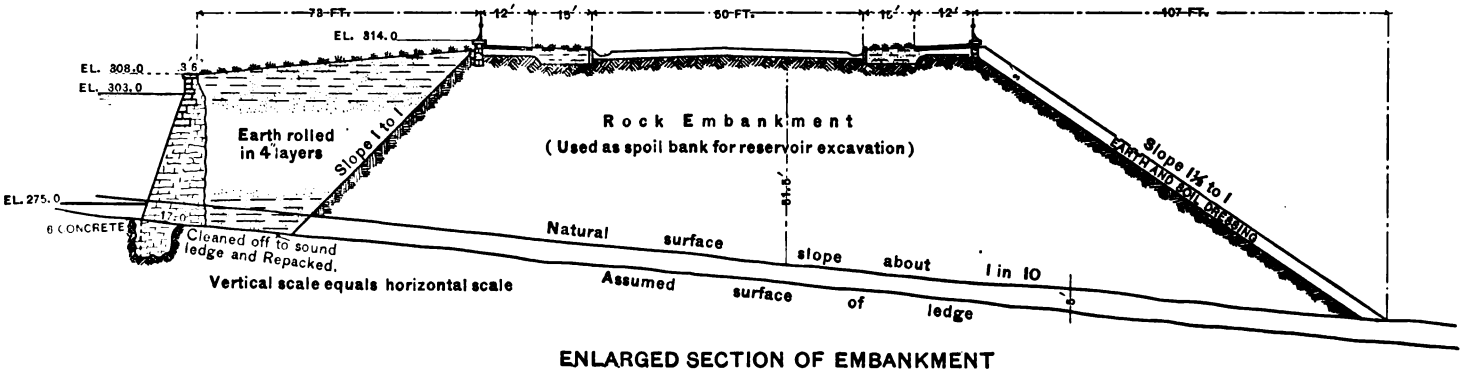
PARK HILL RESERVOIR

Provisional Design for Purpose of Preliminary Estimate



CROSS SECTION OF NORTH BASIN ON A-A.

Embankment to be proportioned so as to balance the excavation.



ENLARGED SECTION OF EMBANKMENT

SHOWING PROPOSED METHOD OF CONSTRUCTION, ALSO MARGINAL AVENUE

(Provisional Design for Preliminary Estimate)

No. 92

TERMINAL RESERVOIR AT PARK HILL.

A large terminal reservoir for equalizing the hourly variations in rate of consumption and permitting the aqueduct to deliver at a uniform rate, and which shall furthermore at all times hold a reserve sufficient for about two days' supply during interruption of aqueduct flow is a necessity with any system of long distance high-pressure supply. This Park Hill Reservoir will serve equally well for the Housatonic, Wallkill, Esopus, Hudson Filtration, or Adirondack source, and no other equally advantageous ground is found.

Ground at a suitable elevation and of comparatively large area is found about one-half mile north of the city limits, suitable for a terminal reservoir adapted to store water ready for distribution at an elevation of 300 feet above sea level and of a capacity nearly as large as that of the reservoir now under construction at Jerome Park. The area of ground at suitable height would permit a total area of water surface nearly double that in the large Central Park Reservoir and the total contents can obviously be varied according to the depth of excavation. This land now lies vacant and is mostly covered by woods and bushes, although a few streets have been platted across a portion of it. A topographical survey by stadia, with contour elevations based upon an adjacent bench-mark of the Croton Aqueduct Commission, was made by Messrs. Ropes and Taber and the principal lines are reproduced in Figure 91 The method proposed provisionally as a basis of estimate for the construction of this reservoir, is given in Figures Nos. 91 and 92.

No test-pits were sunk, but it is believed that the assumptions made as to depth of ledge below surface are on the safe side, and the character of the construction with its enormous masonry walls, while of a proper type for this location, is much more expensive than a type that could be readily devised.

In this provisional design it is proposed to at first develop only the northerly of the two basins, thus obtaining a basin of 790 million gallons capacity and 84 acres water surface. This capacity is about 80 per cent. of the capacity of the large basin in Central Park and, at the present rate of consumption, would supply the whole of Manhattan and The Bronx for about three days, thus permitting the aqueduct line between Rye and Park Hill to be emptied for one or two days for purposes of cleaning, once each six months. The estimated cost of conduits includes a by-pass arrangement, permitting the city to be supplied direct from the aqueduct without the water passing through this basin.

It will be noted from Figure 92 that in this provisional design the margin of the reservoir site has been used for a spoil-bank, thus permitting the construction of a very sightly pleasure drive about the reservoir and at the same

time affording an embankment so massive and of such a water-resisting character as to remove any possible apprehensions of disaster by the bursting of the reservoir bank and the escape of a torrent; the entire outer embankment being in fact a porous rock fill dam—the safest type known for resistance to sudden rupture. It will be noted that as a safeguard against leakage due to temperature cracks in the masonry the provisional design and estimate of cost provide for a very perfect earth fill, rolled in thin layers outside of the masonry.

POSSIBLE FILTERS AT PARK HILL.

A project for utilizing this Park Hill site for both storage and filtration was in part outlined, but time was not available for working it up properly for presentation. This suggestion of planning for the future installation of filters at some point along the conduit lines, is in line with the statement made elsewhere in this report, that the tendency of the times is plainly toward a higher standard of purity for public water supplies and that *the cost of filtration is easily within the means of the New York Department of Water Supply without an expense that would be burdensome to the consumers*, particularly if good progress in waste prevention is made during the next 10 or 20 years. It is not yet certain where the most favorable location for a filtration plant would be found, and the necessity for storing water from the Housatonic, or any of the other possible sources, in *covered* distributing reservoirs perhaps awaits further investigation before it can be considered as settled. If it proves, as appears probable, that *covered* terminal reservoirs will be a necessity with filtered water, then there are possibilities of economy in locating the filters as near to the equalizing reservoir at the terminus as is practicable.

It is possible that certain of the basins forming a filter plant, if located at this point, could be utilized for the four purposes of emergency storage, storage for equalizing night and day draft, for filter basins and filtered water reservoir; the "raw" water being drawn upon in case of great emergency.

It is therefore recommended that in any design for a reservoir at this point, the possibility of filtration being demanded twenty years hence and the comparative cost of locating a part of the filter beds here at Park Hill, without any pumping whatever, should be considered. The total area at this hill-top now at suitable elevation, together with the added area that could be gained by proper distribution of the material excavated in forming the reservoirs, would aggregate more than 200 acres.

The unit prices for the larger items in the following estimates of cost of proposed Park Hill Reservoir are substantially the same as those under which the work of Jerome Park Reservoir is being performed. The cost for acquiring this land, as entered, is an assumption, but one which appears more

generous when one inspects the site. No reliable figures from actual sales in large quantity were learned, and if the land is taken by eminent domain, it is hard to say what the city might have to pay.

TABLE No. 59.

Cost of Park Hill Reservoir—Rough Preliminary Estimate.

	North Basin.	*South Basin.*
Area of water surface	84 acres.	80 acres.
Capacity	790,000,000 gallons.	730,000,000 gallons.
Shore line	6,300 feet.	7,400 feet.
Division wall	1,700 feet.	1,300 feet.

Item.	Unit Price.	Quantity.	Amount.	Quantity.	Amount.
Earth excavation	$0 25	728,000 cu. yds.	$182,000	600,000 cu. yds.	$150,000
Rock excavation	90	1,088,000 "	980,000	946,000 "	851,000
Rock excavation in trench	5 00	13,900 "	70,000	16,300 "	82,000
Soil dressing and sodding	40	185,000 "	74,000	217,000 "	87,000
Earth embankment, hauled, spread and rolled	15	495,000 "	74,000	524,000 "	79,000
Reservoir wall, 33 feet high	78 00	7,430 lin. ft.	580,000	8,270 lin. ft.	645,000
Division wall of less height	48 00	570 "	27,000	430 "	21,000
Portland concrete lining	6 00	67,800 cu. yds.	407,000	64,500 cu. yds.	387,000
Gate-houses	.....	2	200,000	2 enlarged, 1 additional.	150,000
Marginal avenues, with footwalks, curbing, gutters, etc.	10 20	10,300 lin. ft.	105,000	8,900 lin. ft.	91,000
Viaduct over Yonkers avenue	.....	.....	118,000	.....	.....
Cast and wrought iron railing	4 00	18,100 lin. ft.	72,000	18,500 lin. ft.	74,000
Underground drainage system for roadways and embankments	.....	.....	39,000	.....	46,000
Acres of land, buildings, etc.—					
North (assumed)	2,000 00	143 acres.	286,000	148 acres.	252,000
South (assumed)	1,700 00				
			$3,214,000		$2,915,000
Add 15 per cent. for contingencies and supervision	.....	.....	482,000	.....	437,000
Total	.....	.....	$3,696,000	.....	$3,352,000

Park Hill Reservoir—Summary of Cost.

	Cost of Construction.		Present Value.
North Basin	$3,696,000		$3,696,000
South Basin	3,352,000	If deferred 20 years	1,856,000
Total			$5,542,000

Both basins—

Area of water surface, 164 acres.

Capacity, 1,520,000,000 gallons. Can be increased by blasting deeper or building higher walls or taking slightly larger area.

Appendix No. 8.

WATER-POWER DAMAGES.

Injury to Water-Powers on the Housatonic River that Would be Caused by the Diversion of Ten Mile River.

The diversion of the Ten Mile river, without taking any part of the Housatonic, will destroy three small water-powers at Amenia, Dover Plains and South Dover, in all aggregating about 150 horse-power at ordinary summer flow. Along the lower Housatonic river in Connecticut, the diversion of the Ten Mile will take away from 12 to 18 per cent. of the drainage area now supplying Derby, Lanesville and New Milford, at which points are the only mill sites now developed along the stream; the effect being to take away about one-sixth to one-eighth part of the entire water-power (a part of which is not yet in use), at ordinary summer flow. This diversion will also take away about 15 per cent. from the value of the opportunity to build dams and develop power at five other proposed dam sites on the lower Housatonic.

Whether we follow the former Brooklyn plan for developing the Ten Mile river supply in three independent basins, or follow the plan herein proposed for a single large dam on the Ten Mile river near Webatuck, the damage caused to water-powers all along the Ten Mile and Housatonic rivers will be substantially the same. Precisely the same mill sites both developed and undeveloped would be affected, the only difference being that with the Brooklyn Dam location about 10 per cent. less drainage area, and consequently, about 10 per cent. less water, would be taken away than by a dam near Webatuck; while with the Ten Mile River Dam located at the State line, about 11 per cent. more drainage area and about 11 per cent. more water-power would be diverted than for the Webatuck Dam site.

The three small mills within the Ten Mile Valley would have their water-powers destroyed just about as effectively by a dam at one site as at the other.

Abandoned Water Privileges on the Ten Mile River.

In early days when iron brought higher prices, and when the forests afforded cheap charcoal, since ore was abundant in this region, water-power for blowing the small iron furnaces was in great demand, and many small powers were developed in a crude way which to-day it would not pay to maintain. We note the following locations where water-power has been developed and abandoned. So far as could be learned there are no others that would be affected in any way by the proposed diversion.

1st. At State line abandoned about twenty-five years ago.
2d. At Webatuck. Saw-mill abandoned about eight years ago.
3d. Dover Furnace on Swamp river, abandoned about forty years ago.
4th. At Dover Furnace on brook leading into Swamp river, abandoned about thirty years ago.
5th. At Steel Works settlement in Town of Amenia, last used by a grist-mill, abandoned about five years ago.
6th. At Singpak, in the Town of Amenia, abandoned thirty years ago.

These water-powers were all small, and not one of the above locations has any apparent commercial value as a mill site to-day.

Developed Water-Powers.

The only developed water-powers upon the Ten Mile river are at the sites named below.

All that appeared to possess any importance were visited by Morris Knowles, C. E., who collected most of the local data upon which I have based the following preliminary estimates:

1st. A small country grist-mill at South Dover, using under favorable conditions probably about 75 to 100 horse-power under 9 feet gross average fall. Water supplied from 200 square miles of watershed without storage. Apparently no opportunity to increase the present fall materially; mill pond holds perhaps at most 1,500,000 cubic feet storage by drawing 1 foot. Day increase from night storage possibly 20 to 40 cubic feet per second. This is an ordinary country grist-mill, which grinds corn for the neighboring farmers. It has 4 turbines—probably 2 are of the scroll pattern. In dry weather there is only water sufficient for 2 turbines.

2d. A grist-mill and cider-mill and an electric light plant at Dover Plains, using in all a daily average of about 50 horse-power, under 8 feet gross fall, and yielding 100 horse-power at most for a few hours when furnishing electric lights. Water from 134 square miles of watershed without storage. Mill-pond storage probably not over 800,000 cubic feet by drawing 1 foot. Day increase from night storage possibly 15 to 25 cubic feet second. The electric lighting plant is said to be owned by Adolph Reimer, of Brooklyn, N. Y. The grist-mill by A. H. Hoffman, of Dover Plains. The electric plant has a turbine that is said to be of 60 horse-power. It lights the village

streets and some stores and dwellings, begins late in afternoon and runs until late in evening, when there is water enough. There is no steam auxiliary, and in drought of October, 1899, it was noted that village was without electric lights several nights each week. The grist-mill has little occasion to run in the dry season.

3d. Small grist-mill at South Amenia, using probably not over 20 or 30 horse-power, under 8 feet fall. Water from 80 square miles of watershed. Mill pond storage by night for day use probably small.

The only developed water-powers on the Housatonic below the confluence of the Ten Mile river, and, therefore, affected by its diversion, are the three named below:

1st. At Derby (Birmingham), the Ousatonic Power Company, using in all not far from 2,000 horse-power by day, and perhaps 500 horse-power at night.

2d. At Lanesville, the Bridgeport Wood Finishing Company, 2½ miles below New Milford, Conn., grinding silex for paint and "wood-filling," using at present in all not far from 250 horse-power.

3d. New Milford site, the small electric-light plant of Mr. Levi P. Giddings, together with a power plant, from which he leases power to pottery. The whole power used at present being from 150 to 175 horse-power.

Undeveloped Dam and Power Sites of Which Development Is Now Proposed.

There are six other mill sites on the Housatonic river where water diversion damages must be considered. Five of these mill sites are below the confluence of the Ten Mile with the Housatonic and one just above it. These developments are proposed by two new companies, both chartered in 1893, neither of which as yet has any power plant or manufacturing plant. The two companies have different managers, but appear to be working along the same general lines. Each is reported to have been for two or three years past buying up such small, isolated pieces of land controlling the possible dam sites, as could be had at a moderate price.

There are no other water-power sites below the confluence of the Ten Mile with the Housatonic that have ever been developed, and so far as I can learn, no others which it is proposed to develop.

Dam Sites Proposed by the Housatonic Power Company.

Three locations: 1st. Zoar; 2d. Little York; 3d. Southville.

This Company is said to be headed by Mr. A. M. Young, of No. 100 Broadway, New York, a gentleman prominently identified with electric lighting plants and electric traction companies in various cities of Southwestern Connecticut; and has received among others the following special privileges from the Connecticut Legislature:

Acts and Resolves, 1893; page 111. Housatonic Power Company chartered to build 30-foot dam between Jackson Place and Ousatonic Company's mill pond (Zoar site). Incorporators, Thomas Wallace, Thomas L. Cornell, Robert S. Hunniman, D. S. Brinsmade, R. F. Gaylord, Allen W. Paige.

Acts, 1895; page 247; Charter amended to permit conveying power from Southville in Bridgewater by wire to New Haven County.

Acts 1899, page 23; Charter amended to allow dam of reasonable height at Southville, also right to build dam at Little York, below Southville.

Dam Sites Proposed by the New Milford Power Company.

Three locations: 1st. New Bridge; 2d. Gaylordsville; 3d. Bulls Bridge.

This Company has received the following special privileges from the Connecticut Legislature:

Conn. Acts and Resolves, 1893; page 288. Charter granted to incorporators, Nicholas Staub, Frank C. Sumner, Louis F. Heublin, Joseph Hirth, Allen W. Paige. Right to build dam from one to three miles below Lover's Leap Falls in New Milford to height within 5 feet of level of waterwheel platform of Bridgeport Wood Finishing Company.

Acts of 1895, page 28. Charter amended, giving right to build dam at Gaylordsville and another at Bulls Bridge (Bulls Bridge is half a mile *above* the outlet of the Ten Mile river).

Injury to the Developed Water-power at Birmingham by Diversion of the Entire Ten Mile River.

The Ousatonic Power Company, a large enterprise supplying various factories at Derby, Conn., with water for power, would lose the drainage from 200 square miles of watershed out of its present 1,580 square miles, which is about one-eighth part of the whole, and therefore about one-eighth part of its entire power would be taken away. This Company states that it has not yet

sold or leased its entire power, so perhaps the diversion would not seriously affect the permanent power rights of its present lessees.

The U. S. Census Report of 1880 on Water Power, Volume 1, page 311, gives a very full description of this plant, and quotes the rates of rental received for power. This Census Report estimates the watershed at 1,562 square miles, and the head or fall of water, which varies some 3 or 4 feet with rise and fall of tide, is stated to average 22 feet.

By planimeter from the later maps of the U. S. Geological Survey we find the drainage area to be 1,580 square miles. The Census Report states that careful measurement of flow of river had been made up to 1880, and presumably have been continued ever since in the form of daily heights of water. Page 308 of Census Report states that during the development of the power (1864–1870 ?) measurements were made which indicated that 500 cubic feet per second could be relied upon for the 24 hour flow in time of drought. This quantity was still relied upon in 1882, and Mr. Brinsmade, Chief Engineer of the Company, tells us in December, 1899, that the past season has been as severe as any, and confirms the early estimate, and says that notwithstanding the severity of the drought, they have not had to cut off any of their permanent power.

The total cost of this water-power development prior to the construction of new canal is said to have been $430,000. The new canal and other improvements to January 1, 1900, do not appear to be such as would bring the total cost up to date above $500,000.

In the studies of Mr. Horace Loomis in 1878 for the water supply of New York from the Housatonic, the flow was gauged at Kent from about 755 square miles of watershed. The methods and precision of measurement are not stated. The minimum flow noted in these observations extending from June to October was 260 cubic feet per second = 0.34 cubic feet per second per square mile; the average flow for five months was 460 cubic feet per second = 0.61 cubic feet per second per square mile. The above minimum figure of Mr. Brinsmade, 500 cubic feet per second from 1,580 square miles gives 0.316 cubic feet per second per square mile.

According to the observed flows of the Croton river and of the Connecticut river, this summer season of 1878, when the Loomis gaugings were made, was a little more wet than the average summer, as appears by following comparison:

TABLE No. 60.

Comparison of Flow in Cubic Feet per Second per Square Mile in Period Containing the Loomis Gaugings.

	Croton 1878, with evaporating conditions as in 1902.	Average Croton 32 Years.	Connecticut 1878.	Average Connecticut 13 Years.
Driest month.....	.44	.26	.67	.64
2d "	.51	.41	.74	.68
3d "	.73	.55	.75	.74
4th "	1.13	.74	.95	.86
5th "	1.17	.93	1.30	1.04
Average	.80	.58	.88	.79

From the Census Report we further learn that total power in use up to 1882 was 1,500 horse-power by day, 500 horse-power by night, and that during extreme droughts these amounts could not then be furnished, but it was believed that these amounts of power could be furnished in future by means of more flashboard storage. In 1899, Mr. Brinsmade informs us that more power has since been sold and that more is now for sale, but is not ready to state exact figures. The Census Report further states, page 312, that price received for this water-power is as follows, taking efficiency of turbines 75 per cent.:

"*Permanent power*," $26.67 per net horse-power per year.

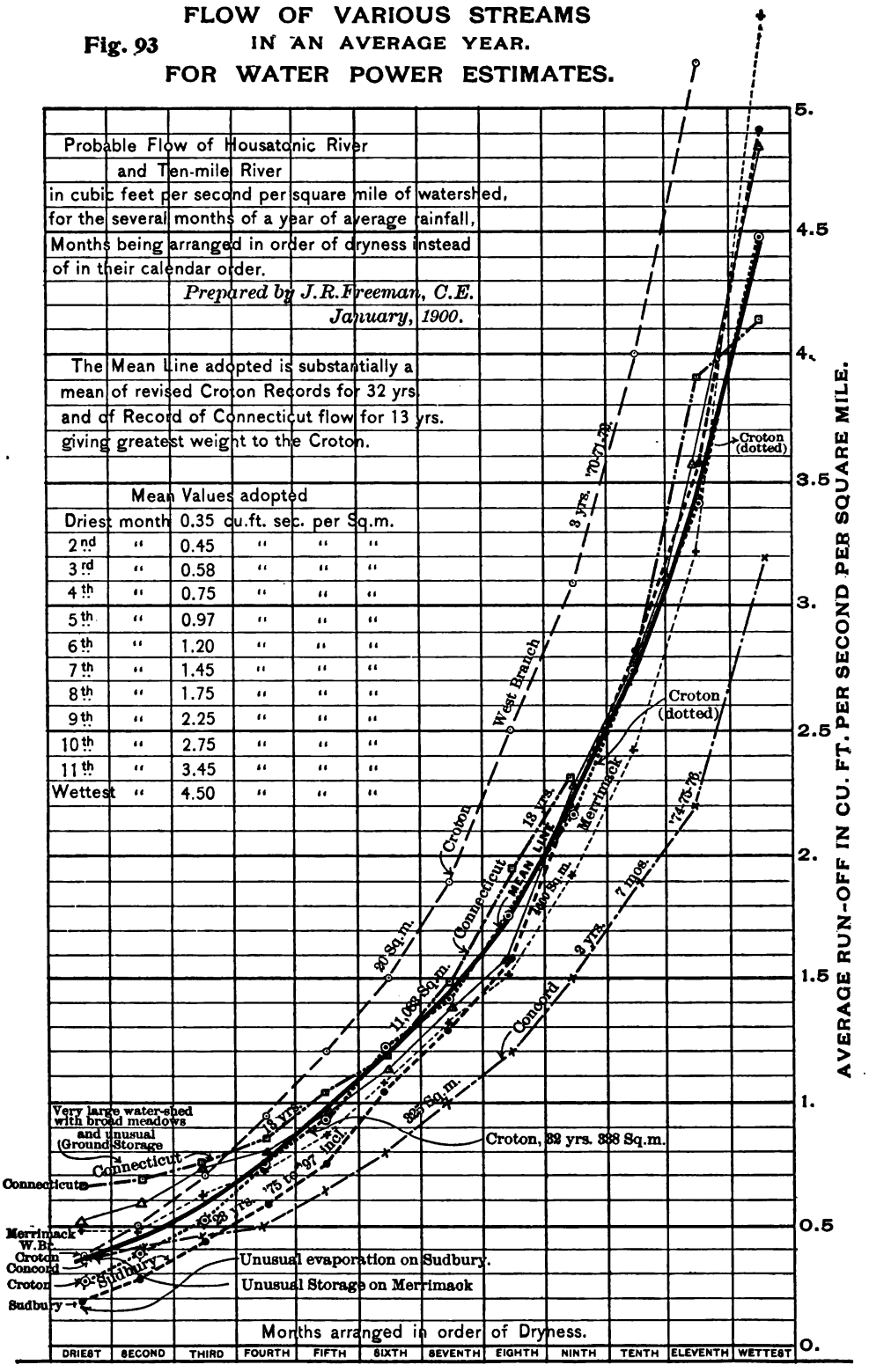
"*First surplus*" or intermittent power, $16 per net horse-power per year.

"*Second surplus*" or intermittent power, for 12 hours a day, 6 days per week, $10.67 per net horse-power per year.

Probable Flow of Housatonic River in a Year of Average Rainfall.

The following diagram gives the flow of five rivers in New York and New England which have been accurately gauged for a term of years. From these records we can deduce the flow of the Housatonic much more accurately than from the brief period of gaugings of Mr. Loomis, or the statements quoted in the Census Report.

FLOW OF VARIOUS STREAMS
Fig. 93 IN AN AVERAGE YEAR.
FOR WATER POWER ESTIMATES.



We are informed that the Ousatonic Power Company has daily records of heights of water wasting over its dam, from which, in connection with such records as may exist of daily power drawn by lessees, a fairly accurate estimate of the annual yield could probably be made. We are, however, informed that these observations have not yet been worked up. In the following computations the line adopted for the Housatonic flow per square mile in the various months of the average year is a mean between the revised natural flow of the Croton river and the flow of the Connecticut.

TABLE NO. 61.

Estimate of Largest Practicable Power Development at Birmingham, Conn., and of Impairment by Diversion of the Ten Mile River only, in a Year of Average Rainfall.

Proportion of water taken away by diversion of entire Ten Mile, one-eighth of whole.

Month in Order of Dryness.	24-hour 7 day Flow in Cu. Ft. per Sec. per Sq. Mile. (From Average Croton and Connecticut River Experience and Others.)	24-hour 7-day Flow at Birmingham, 1,580 Sq. miles. Cu. Ft. Sec.	Probable Quantity that can be Added to the Day-flow by Night and Sunday Storage. Cu. Ft. S.	Probable Maximum Average Day-flow by Use of Night and Sunday Storage. Cu. Ft. S.	Probable Average Net Day Horse Power with Maximum Development. 21 Feet Net Average Fall— 75% Effic.— Col. 5 x 1.785 Horse Power.	Maximum 12-hour 6-day Power Taken Away by Diversion of 200 Sq. Miles. Ten Mile Watershed. $\frac{200}{1580}$=12.65% of Col. 6 Horse Power.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Dryest	.35	550	370	920	1,640	208
2d	.45	710	370	1,080	1,930	244
3d	.58	915	320	1,235	2,200	278
4th	.75	1,185	200	1,385	2,470	312
5th	.97	1,530	70	1,600	2,860	362
6th	1.20	1,900	...	1,900	3,390	400
7th	1.45	2,300	...	2,300	4,110	300
8th	1.75					Ave. 300
9th	2.25					
10th	2.75					
11th	3.45					
12th	4.50					

Note that above minimum flow in dryest month of average year is 10 per cent. greater than the minimum by the Brinsmade and the Loomis gaugings.

Night storage assumed equivalent to 2 feet depth on pond of 8,000,000 square feet area = 16,000,000 cubic feet which, if all drawn in 12 hours, would yield 370 cubic feet per second for 12 hours.

The entire average flow cannot be realized from use of water-power with limited storage, because in sudden heavy rains there must be a considerable volume escape over dam in months when the *average flow* is below the total turbine capacity.

At Birmingham the maximum impairment that can be caused by the diversion of the Ten Mile river estimated on most liberal reasonable basis is 300 horse-power for 7 months of the year.

Under similar circumstances the Metropolitan Water Board, because of the diversion of the Nashua river for Boston's supply in 1896, paid to the Essex Water Power Company of Lawrence, Mass., for the diversion of 118.23 square miles of watershed having very much better storage than the Housatonic and used under 32 feet fall at Lawrence, the sum of $87,000. In the same proportion, having regard to the watershed area of the Ten Mile and the net fall at Birmingham, the fair award to the Ousatonic Power Company would be $97,000 for the diversion of $\frac{1}{8}$ part of their watershed.

At Lanesville, New Milford, Conn.

The Bridgeport Wood Finishing Company grinds silex (quartz) to make "filling" for use in hardwood finishing, paint, soap, etc.

The drainage area above this mill site is 1,198 square miles, and there is $8\frac{1}{2}$ feet fall. By the eye, without surveys, it appears impracticable to materially increase this fall. The power is now used 24 hours a day. The circumstances are not favorable to concentrating this water all for day use and storing the night flow, because the pond storage is limited to the river above the dam for about two miles in length by, say, 150 feet wide. Any considerable amount of storage is impracticable without drawing down the pond so much in the afternoon as to materially lessen the fall.

Mill is stated to be now using 250 horse-power. Uses no steam. Have two 66-inch Rodney Hunt wheels, of a model probably giving 125 horse-power each, under $8\frac{1}{2}$ feet fall. This site is favorable for additional turbines. This privilege if fully developed to same relative turbine capacity as that assumed for the Ousatonic Power Company at Derby, and if it had the same pond capacity for night storage, should give:

$$\frac{1198}{1580} \times \frac{8.5}{22} \times \frac{.70}{.75} = 27 \text{ per cent. of power at Birmingham.}$$

At present this mill privilege is not fully developed, and there is a surplus of water beyond requirements, which runs to waste. Although the diversion of the Ten Mile would not shut down any present machinery, it would impair the opportunity to develop the power so fully as otherwise. Since this mill runs 24 hours, the utmost reasonable development would be up to the flow

of the fourth month, or to 570 net horse-power before diversion (see table following).

About 1,000 feet downstream from this Lanesville mill is Lover's Leap Fall, of perhaps 15 feet, where it is said the New Milford Power Company, chartered in 1893, proposed to build a dam, but abandoned the project because of fear of back water injury to the privilege of the Bridgeport Wood Finishing Company.

ESTIMATE OF LARGEST PRACTICABLE DEVELOPMENT OF WATER-POWER AT LANESVILLE, CONN.

SITE OWNED BY THE BRIDGEPORT WOOD FINISHING COMPANY.

TABLE NO. 62.

Impairment of Water-Power by Diversion of Ten Mile River Only, in a Year of Average Rainfall.

Proportion of entire watershed taken away by diversion of the entire Ten Mile river at Webatuck.

$$\frac{200}{1198} = \frac{1}{6}$$

Month in Order of Dryness.	24 Hour 7-days' Flow in Cubic Feet per Second per Square Mile.	Total 24 Hour 7-days' Flow of Housatonic at Lanesville. Col. 2 x 1198. Cubic feet per Second.	Net Horse Power 24 Hours a Day, this would give under 8 Feet Net Fall at 70% Efficiency. Col. 3x.635.	Power Taken Away if Fully Developed by Diversion of entire Ten Mile. $\frac{200}{1198}$ x Col. 4.	Probable Present Total Turbine Capacity. Horse Power.	Power Actually Taken Away from Present Development by Diversion of Ten Mile.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Dryest	.35	420	266	44	250	0
2d Dryest	.45	540	343	57	250	0
3d "	.58	695	440	73	250	0
4th "	.75	900	570	95	250	0
5th "	....	....	570	50?	....	..
6th "	....	....	570	Av. 64	....	..
7th "	....	....	570	0	....	..
8th "	....	....	570	0	....	..
9th "	....	....	570	0	....	..
10th "	....	....	570	0	....	..
11th "	....	....	570	0	....	..
12th "	....	....	570	0	....	..

The foregoing table shows that there is no *power* taken away from the present development, but it is the *opportunity* to develop more power at some future time that is impaired. Moreover, since on mill privileges of this class abundant experience proves that it rarely, if ever, pays to put in turbines to carry more than the entire flow of the river in the fourth month in order of dryness, the diversion of the Ten Mile only would not prevent water running to waste over the dam for two-thirds of the time in any ordinary year, and at all such times when water would be wasting, there could be no real injury. Nevertheless, it appears ordinary and proper to adjust damages on a somewhat more liberal basis, so assume that since the drainage area diverted from Lanesville is exactly the same as the drainage area diverted from Derby and the net fall at Lanesville is 8/21 of fall at Derby, the power that the diverted water would give (if developed) might be claimed to be worth 8/21 as much at Lanesville as at Derby, and if we call undeveloped power, needing only more turbines and flumes to develop it, worth 2/3 as much as power already marketed, we get, on the basis figured for Birmingham, $97{,}000 \times 8/21 \times 2/3 \times .70/.75 = \$23{,}000$, as a theoretic value for damages at Lanesville, which is certainly several times what the amount taken away from this opportunity to develop more power is worth commercialy, but is not more than half what could be figured out on the steam substitution basis, using high values for steam and neglecting the disadvantages of a small variable power.

WATER-POWER AT NEW MILFORD.

LEVI P. GIDDINGS POWER AND ELECTRIC-LIGHT PLANT.

Pottery—Lessee in Part.

The flumes and races are short and the turbines are all in one compact group. The drainage area above this mill site is 1,123 square miles. The fall now is 9½ feet gross, probably 9 feet net. It was formerly 7 feet.

The pottery is said to use about 30 horse-power, by day only.

Total power at present in use in maximum hour is claimed to be from 150 to 175 horse-power.

The electric light plant probably runs on an average 6 hours out of the 24, furnishing lights for the streets and some of the residences in New Milford. This power plant has two 72-inch Hunt turbines (probably of an early pattern) and one 60-inch Hunt turbine (possibly of recent model); also one other turbine of a size and make not now remembered by Mr. Giddings. It also has an auxiliary steam plant, consisting of an engine for furnishing

power in time of low water, of about 185 horse-power, with two boilers of about 85 horse-power each. They are said to use this steam plant for furnishing a part of the power for about two weeks in dry years when water is insufficient. All electric power stops at midnight and street lamps are not run on moonlight nights. An additional canal for further development of power for use in wet months is apparently feasible, although not contemplated. The storage capacity in mill pond is not large, and apparently they cannot store the night flow except when very small.

Power factor per cubic foot is $\frac{9 \times 62.4}{550} \times 70$ per cent..... 0.72

TABLE NO. 63.

ESTIMATE OF LARGEST PRACTICABLE DEVELOPMENT OF WATER-POWER AT NEW MILFORD, CONN.

Impairment of Power by Diversion of Ten Mile River Only, in a Year of Average Rainfall.

	C. f. s. per Square Mile, 24-hour, 7-day.	Total 24-hour 7-day Flow at New Milford. — Col. 2 x 1123, C. f. p. s.	24-hour Horse-power. Col. 3 x 0.72.	Actual Present Development Horse-power.	Horse-power taken from Present Development.	Horse-power taken from Maximum Development Reasonably Possible. — $\frac{200}{1123}$ x Col. 4
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Dryest month.....	.35	394 c. f. p. s. x 72.	283	175	0	50
2d dryest month.....	.45	505 "	363	173	0	65
3d "	.58	650 "	467	175	0	83
4th "	.75	843 "	600	175	0	107
5th "	.97	1,090 "	785 000	175	0	50
6th "	...	.....	....	...	..	Av. 71 for 5 mos.

Pond storage at night apparently insufficient to augment day flow materially.

After the diversion of the entire Ten Mile there would probably at all times be more water left than the present turbines could use; hence the injury consists in *impairing the opportunity* to develop power at some future time.

On the same basis as estimated for Lanesville—calling developed power worth the same as at Birmingham, and undeveloped power where dam and factories already exist, worth 2/3 of developed power—since net fall here is

$$\frac{9}{21} \text{ of net average fall on turbines at Birmingham,}$$

$$\$97,000 \times \frac{9}{21} \times 2/3 \times \frac{.70}{.75} = \$25,900.$$

—which theoretic value is, as before found, several times what this impairment of opportunity is worth on any ordinary commercial basis, but is not more than half what it might be figured out on the steam substitution basis and neglecting the disadvantage of a small variable water-power.

Impairment of Opportunity to Develop Power at the Five New Dam Sites Recently Proposed Below the Mouth of the Ten Mile River.

Site at Zoar—Eight miles above Derby Bridge, 50 feet fall proposed. Drainage area 1,532 square miles. The proposition is understood to be for dam 34 feet high, with canal 2 miles long, giving 50 feet gross fall. Proposed for electric transmission. Mill pond would probably flow back to Bennett's Bridge, about 5 miles. The works to give 50 feet fall appear so very expensive that there is a question if this enterprise can be made to show sufficient return to warrant its construction, but if it is to be actually developed, the cost of land, dams and canals would be the same, whether the Ten Mile river is to be diverted or not; that is, the owners would save nothing substantial in the cost of development, but would lose one-eighth of the possible power.

Site at Little York—Eighteen miles above Derby Bridge, fall now about 8 feet. Proposed fall not stated; perhaps 12 feet can be secured. Watershed above here 1,389 square miles. There was once a saw-mill here, but it was carried off in a freshet forty-five years ago, and dam site on main river since abandoned. A grist-mill near here takes water from a small side stream. Appearance of site, the probability of back-water and the low fall make it open to serious question if this power can ever be profitably developed. Housatonic Company said not to own the old mill site, but are said to have options on west bank of river only.

Site at Southville, Babbitt's Bridge—Drainage area above here 1,219 square miles. Twenty-two feet gross fall, as proposed. Abandoned as a mill site for past forty-five years. At various times before this there was, at this site, a saw-mill, grist-mill, cotton-mill and a hat shop. Dam carried away in freshet of 1854. Housatonic Power Company said to hold options on con-

siderable property here. Canal proposed about 2,000 feet long, giving perhaps 25 feet gross fall.

Site at New Bridge—About 25 miles above Derby Bridge, 4 miles below New Milford. Drainage area 1,206 square miles. Fall proposed probably about 20 feet. Charter gives right to build at a location from 1 to 3 miles below Lover's Leap Falls. Could not learn that definite options on controlling lands had been secured.

Site at Gaylordsville—Drainage area 998 square miles. Former dam here washed away about twenty-five years ago—not rebuilt. Said to have been 7 feet high. Apparently can get 15 or 20 feet in all by canal 1,500 or 2,000 feet in length. Company said to have not yet secured options on all controlling lands.

Possible Damages at Undeveloped Sites.

It is extremely difficult to estimate what damages might be claimed or awarded in connection with a diversion of water from sites which have long been abandoned and are not now developed, but are merely "prospects" with owners trying to convince capitalists that they are worth the cost of development.

As illustrating the recent commercial value of lands controlling these prospective power sites, it is reported that at the proposed Gaylordsville site the lot of ¾ acre containing the site of the west abutment of the former dam and the former mill site on the west side of river, was purchased by parties representing the New Milford Power Company for $700, and the company declined to give $1,000 for the corresponding lot on the east side of river; but, of the accuracy of this statement I have no certain knowledge. If true, this furnishes no criterion of the claims for damage after mechanical and commercial genius has transformed waste land into a power site, and in forecasting the *possible cost to the City of New York*, the only prudent way is to estimate upon the possibility of these being viewed as actual water-powers.

The total fall proposed at these five possible dam sites and the present watershed areas are about as follows:

Zoar	50 ft. fall x 1,532	square miles drainage.
Little York	12 ft. fall x 1,389	" " "
Southville	25 ft. fall x 1,219	" " "
New Bridge	20 ft. fall x 1,206	" " "
Gaylordsville	15 ft. fall x 998	" " "
Total, about	122 ft. fall.	

The whole is thus equivalent to about 122 feet gross fall, or about 117 feet net fall with a drainage area averaging 1,269 square miles.

In the cases of the water-powers at Lowell and Lawrence in 1896 the Metropolitan Water Board (of which I was a member, and thus familiar with all the facts), acting for the Commonwealth of Massachusetts, arrived at an amicable and satisfactory settlement with the owners for power actually taken away by the diversion of the South Branch of the Nashua river. These two water-powers at Lowell and Lawrence were developed with the best of granite dams and substantial canals, all in the very best of order, and in actual use in two of the chief manufacturing cities of New England. The price agreed upon was about $200,000. This covered the diversion of about 118 square miles of watershed provided with storage reservoirs much more ample than those on the Ten Mile watershed, and the total fall was about 72 feet.

In the same proportion, the damage by diverting the 200 square miles of the Ten Mile Watershed from a total fall of 122 feet would be

$$\$200{,}000 \times \frac{200 \text{ square miles}}{118 \text{ square miles}} \times \frac{122 \text{ feet fall}}{72 \text{ feet fall}} = \$575{,}000.$$

Surely, under the most liberal view, the damage, as measured by impairment of fair value on these *mere opportunities* or *possibilities* for developing power in the woods, should be only a fraction of this sum paid for *power actually developed* and largely in use in centres of industry like Lowell and Lawrence, because a large sum must be expended before any power could be had at these unused mill sites.

WATER-POWER DAMAGES, TEN MILE AND UPPER HOUSATONIC.

Injury to Water-powers on the Housatonic River in the State of Connecticut that Would Be Caused by the Diversion of the Upper Housatonic and the Ten Mile Rivers.

It would be 15 years hence before the *ordinary flow* of the river below Merwinsville would need to be affected by any diversion for New York. Prior to that time by merely storing and *using flood water, which now runs to waste*, sufficient water could be obtained for all of New York's needs, without in the slightest degree impairing the water-power of any mill between New Milford and the sea.

The custom of the law has been to treat such diversions as immediate and entire, but as this is contrary to the fact, I believe it could be changed and a charter drawn which should until the year 1915 require the present rate of

summer flow maintained and then permit a further progressive diversion afterward until all was taken—so that damages could be adjusted in accordance with the real needs. The following estimate will, however, proceed on the basis of entire diversion immediately.

The injury that would be caused by the diversion of the Ten Mile only has already been considered. The injury to power sites and mills located along the Ten Mile river is not increased by the diversion of the Upper Housatonic, but to the mills on the Housatonic there comes a much greater injury than that arising from the diversion of the Ten Mile alone because of the four-fold additional volume of water diverted, and because the projected development at Bull's Bridge, which is not interfered with by the diversion of the Ten Mile river, becomes submerged by the proposed Housatonic Dam.

There are also three very small developed mill sites—one cider mill and two small saw and grist-mills, on side streams entering above the proposed large dam that would be submerged.

There are also abandoned mill sites on the main river above Bull's Bridge at Swift Bridge, Kent Furnace and Cornwall Bridge that would be submerged by the proposed Housatonic Dam.

No reason appears why any quantity of water will need to be left steadily flowing in the stream past the proposed Housatonic Dam in dry weather at all future times, as is sometimes the case for "wash-water" or for the dilution of sewage where mills or cities are located close below the point of diversion of a large stream. There will, after diversion, be 105 square miles of watershed left tributary to the stream before it passes New Milford, the nearest village below the proposed diversion; several good-sized tributaries coming in between the proposed dam site and this village giving ample water for dilution of New Milford's sewage, or for all purposes except power at the Gidding's Mill site. By giving to the owner of the mill using water-power a sum sufficient to buy and erect steam engines and boilers, and by the further payment of a sum, the interest on which would buy coal, or, indeed, if given the full commercial value of his present entire plant and paid a fair bonus in addition, he surely can have his injury repaired.

On page 422 of this Appendix will be found a brief description of all the developed and undeveloped powers below the proposed dam site on the Housatonic; we will review them again with the larger diversion in view, beginning at the downstream end.

ESTIMATE OF THE LARGEST PRACTICABLE DEVELOPMENT OF WATER-POWER BY THE OUSATONIC POWER COMPANY AT BIRMINGHAM, CONN.

TABLE NO. 64.

Impairment of Power Caused by the Entire Diversion of the Upper Housatonic and Ten Mile Rivers, in a Year of Average Rainfall.

In table below all is figured as 12-hour power.

Month in order of dryness.	Probable flow in a year of average rain fall. Cubic feet per second per square mile.	24 hour 7 day flow at Birmingham before diversion. 1,580 square miles.	Probable maximum average day flow that can be utilized by use of night and Sunday storage before diversion.	Probable average net day horse-power with maximum development, 21 foot fall before diversion.	24 hour 7 day flow after diversion. 562 square miles.	Probable maximum 12 hour day flow that can be utilized by night and Sunday storage after diversion.	Total of developed and undeveloped power taken away. Horse-power.	Probable average net horse-power left after diversion, 12 hour day.
Dryest	0.35	550	920	1,640	196	331	1,050	590
2d dryest	.45	710	1,080	1,930	253	547	950	980
3d "	.58	915	1,235	2,200	325	678	990	1,210
4th "	.75	1,185	1,385	2,470	420	790	1,060	1,410
5th "	.97	1,530	1,600	2,860	545	915	1,230	1,630
6th "	1.20	1,900	1,900	3,390	675	1,045	1,520	1,870
7th "	1.45	2,300	2,300	4,110	820	1,190	1,990	2,120
8th "	1.75	.....	.....	4,110	985	1,250	1,880	2,230
9th "	2.25	.....	.....	4,110	1,260	1,500	1,430	2,680
10th "	2.75	.....	.....	4,110	1,540	1,540	1,360	2,750
11th "	3.45	.....	.....	4,110	1,940	1,940	650	3,460
12th "	4.50	.....	.....	4,110	2,530	2,000	540	3,570
Average net horse-power taken away for year.....							1,220	.....

According to U. S. Census Report this power leases for from $26.67 down to $10.67 per net horse-power per year. Of this 1,220 horse-power, say two-thirds would be permanent power. Call average price for power taken, $23 per horse-power per year.

There are many large steam power plants to-day in operation in New England, which, including all cost of labor, coal supplies, repairs, interest and depreciation, get uniform permanent steam power at a cost little if any greater per year than this $23 per net horse-power.

Loss of rental if all the present power were leased (it is said that only half the above amount is leased), $1,220 \times \$23 = \$28,100$ per year. This capitalized at 5 per cent. is $562,000 for the maximum power diverted, which is

more than the amount that the entire power development at Birmingham is supposed to have cost, although for seven or eight months in the year after the total diversion of the upper river there would be left flowing in the stream water enough to furnish as much power as the total now in use and nearly 600 net horse-power left in the driest month.

To capitalize the total rental is wrong, because it assumes the entire power already sold and in use at time of development, whereas customers have in thirty years, since dam was built, only reached requirements for about half the amount of power which I have estimated above as possbily available, and this large figure moreover does not recognize the cost which has to be deducted from gross rental received to cover maintenance before net profit is reached.

The settlements made with the Water Power Companies at Lowell and Lawrence by the Massachusetts Metropolitan Water Board, and which from intimate personal knowledge I believe to have been fair to both parties, for an impairment proportionally less serious and for fewer months in the year, but for power not fully in use, would, when reduced to same basis of watershed and fall, amount to about

$$\frac{1,018}{118} \times \frac{21}{72} \times \$200,000 = \$503,000.$$

But, on the other hand, by the steam substitution basis, using high cost for steam and neglecting certain disadvantages of water-power, the total would figure out much more than even this large sum.

Bridgeport Wood Finishing Company, at Lanesville.

This company now has water-power from.....	1,198	square miles.
The watershed diverted would be.....	1,018	" "
Leaving drainage from.....	180	" "

Which, in the distant future, say thirty years hence, when all the water of the Housatonic and Ten Mile was diverted to New York, would leave a river flowing here about as large as the Ten Mile river as it now runs at the South Dover Grist-mill, and, therefore, abundant for all purposes except power, and capable of yielding power to the extent of 75 to 100 horse-power, or about one-third power enough for the present machinery of the Wood Finishing Company.

This factory would naturally remain here, because the silex which it grinds is mined close at hand and the railroad facilities are good, with a siding into the yard.

The natural method for the mill to repair the damage caused by the diversion would therefore be by replacing the diverted water-power with steam about 15 years hence. The plant runs twenty-four hours a day.

ESTIMATE OF THE LARGEST PRACTICABLE DEVELOPMENT OF WATER-POWER AT LANESVILLE SITE OF BRIDGEPORT WOOD FINISHING CO.

TABLE NO. 65.

Impairment of Power Caused by Entire Diversion of the Upper Housatonic and Ten Mile Rivers, in a Year of Average Rainfall.

	Before Diversion.				After Diversion.			
Month in order of dryness.	24-hour 7-day flow in cubic feet per second per square mile.	Total 24-hour 7-day flow of river at Lanesville before diversion, 1,108 square miles.	Total 24-hour net horse-power that could be reasonably developed here, 70% off. 8 feet fall. Col. 3 x .635.	Net horse-power at present developed and used.	Total 24-hour 7-day flow of river, 180 square miles. c. f. s.	Total 24-hour power that could be reasonably developed. Net horse-power. Col. 6 x 635.	Total 24-hour developed power taken away. Net horse-power.	Total additional undeveloped power taken away. Net horse-power.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Dryest.....	0.35	420	266	250	63	40	210	16
2d dryest.....	0.45	540	343	250	81	51	199	93
3d "	0.58	695	440	250	104	66	184	190
4th "	0.75	900	570	250	135	86	164	320
5th "	0.97	....	570	250	175	110	140	320
6th "	1.20	....	570	250	216	137	113	320
7th "	1.45	....	570	250	261	165	85	320
8th "	1.75	....	570	250	315	200	50	320
9th "	2.25	....	570	250	405	257	0	313
10th "	2.75	....	570	250	495	314	0	256
11th "	3.45	....	570	250	620	294	0	176
12th "	4.50	....	570	250	510	514	0	56
							12)1,145 Av. 95	12)2,700 225
Average amount of developed power taken away for the year.....							95 H. P.	.....
Average amount of undeveloped power taken away for the year.....							.....	225 H. P.

Power factor, $\frac{8 \times 62.4}{550} \times 70$ per cent. = 0.635.

Actual awards in similar cases cover an extremely wide range and appear governed more by accident and statement than by commercial value.

If, for a rough preliminary estimate, we call the developed power diverted worth the highest rate charged by the Water Power Company at Derby, $26.67 per horse-power per year, and give to the undeveloped power diverted a present value equal to the lowest rate at Derby, $10.67 per horse-power per year, and deduct 25 per cent. from each figure for cost of care and maintenance of dams, turbines, we have:

Developed power, 95 × $26.67 × 75 per cent	$1,900 per year.
Undeveloped power, 225 × $10.67 × 75 per cent	1,800 " "
Total present value per year	$3,700

This capitalization at 5 per cent. is $74,000

On the steam basis, as sometimes followed, a larger award would be reached.

Six per cent., or indeed, 8 per cent., would be a more just basis than 5 per cent. for capitalization in a case of this kind. I believe that 5 per cent. is the most liberal basis of capitalization that can be properly considered, because the proper basis is a rate of income presenting the same degree of certainty of annual income as the present business. The safety and certainty and rate of yield on investment in a small country water-power is different from an investment in Government bonds and the annual cost to the owner of caring for this kind of water-power investment is much more than that of cutting the coupon from a good bond.

NEW MILFORD SITE.

DIVERSION OF HOUSATONIC AND TEN MILE RIVERS.

Levi P. Giddings Power and Electric Light Plant.

New Milford Site—Diversion of Housatonic and Ten Mile Rivers. Levi P. Giddings Power and Electric Light Plant.

This plant now has a tributary drainage area of....	1,123	square miles.
The drainage area proposed to be diverted.....	1,018	" "
The drainage area remaining.....	105	" "

... The water-power remaining after diversion would therefore be about the same as that at present available at the electric light plant, grist-mill and cider-mill at Dover Plains, which has a little more drainage area and a little less fall.

The water left is thus ample for all purposes except power, and would give more than enough power for the present pottery, if, as reported, this calls for but 30 horse-power. The pottery is not the owner of the power, but a lessee.

The electric light plant could apparently be advantageously located in the village and run by steam, for it is a small affair which it would hardly pay to maintain here if the power used cost as much as steam.

ESTIMATE OF THE LARGEST PRACTICABLE DEVELOPMENT OF WATER-POWER AT NEW MILFORD, CONN.

TABLE NO. 66.

Impairment of Power Caused by Entire Diversion of the Upper Housatonic and Ten Mile Rivers, in a Year of Average Rainfall.

Month in order of dryness.	24-hour 7-day flow in cubic feet per second per square mile.	Total 24-hour 7-day flow of river at Giddings dam before diversion, 1,123 square miles.	Total 24-hour power that could be reasonably developed here. 9 feet fall. Col. 3 × 72.	Power at present developed and used.	Total 24-hour 7-day flow of river from 105 square miles.	Total 24-hour net horse power that could reasonably be developed after diversion. Col. 6 × .72	Total 24-hour net horse-power taken away.	Total additional undeveloped net horse-power taken away.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Dryest	0.35	394	283	175	37	27	148	108
2d dryest	0.45	505	363	175	47	34	141	188
3d "	0.58	650	467	175	61	44	131	292
4th "	0.75	843	600	175	79	57	118	425
5th "	0.97	1,090	600	175	102	73	102	425
6th "	1.20	.....	600	175	126	91	84	425
7th "	1.45	.....	600	175	152	110	65	425
8th "	1.75	.....	600	175	184	132	43	425
9th "	2.25	.....	600	175	236	170	5	425
10th "	2.75	.....	600	175	289	208	0	392
11th "	3.45	.....	600	175	362	260	0	340
12th "	4.50	.....	600	175	472	340	0	260
							12) 837	12) 4,130
							70	344
Average amount of developed power taken away for year							70 H. P.	.....
Average amount of undeveloped power taken away for year							.....	344 H. P.

Power factor $\frac{9 \times 62.4 \times 70\%}{550.} = .72$

If here, as at Lanesville, we value developed power at highest rates charged for water-power at Derby and the undeveloped power at the lowest rates charged at Derby, Conn., and to get net value take off 25 per cent. of gross receipts for care and maintenance, we get—

Developed power—	70 × $26.67 × 75 per cent....	$1,400 per year.
Undeveloped power—	344 × 10.67 × 75 per cent....	2,750 " "
		$4,150

This, capitalized at 5 per cent., for reasons previously stated, gives $83,000, which I believe to be very much more than the real injury to the fair market value of the property by the diversion.

Impairment by Diversion of Housatonic and Ten Mile Rivers.

Six undeveloped water-power sites, at Zoar, Little York, Southville, New Bridge, Gaylordsville, Bull's Bridge.

The falls and the tributary drainage areas before and after diversion are as follows:

	Possible fall (about).	Drainage area before diversion.	Drainage area after diversion.	
1. Zoar	50	1,532	514	Below proposed dam.
2. Little York	12	1,389	371	"
3. Southville	25	1,219	201	"
4. New Bridge	20	1,206	188	"
5. Gaylordsville	15	998	0	Above proposed dam.
6. Bull's Bridge	30	780	0	"
Total	152			

My personal opinion, based upon a somewhat wide acquaintance with water-power development for twenty-five years, is that neither of these sites presents good promise of a fair return on the investment required to develop its power. To present figures in detail would require a close estimate of cost of development and maintenance and of outlay necessary at each plant for auxiliary steam power to use in flood and drought, for the Housatonic is a very variable stream as compared with the Connecticut or Merrimack. Such an estimate I have not had time to make, and it indeed appears unnecessary in the present nebulous state both of the water-power propositions and of securing this source of supply for New York.

It is certain that for very many years past they have had no value whatever for power and have run to waste, and that many of the recent owners have been willing to sell their rights at a very low figure. It is claimed that electrical transmission of power now opens a new field for successful development. Very few, if any, practical applications under similar circumstances can be found to substantiate this claim. I have had occasion as an engineer to analyze several similar propositions and found the cost of development, plus cost of maintenance, larger than any leased output could pay fair interest upon, and that the sites then in question had therefore no real value for power.

There is much popular misapprehension on this question of electric transmission from a variable water-power and failure to understand its lack of economy and other disadvantages, such as necessity for a steam plant in duplicate, to use in flood and drought.

The small prices at which options have been secured on the real estate controlling some of these privileges is of interest. Certain it is that the water at all of the six sites runs to waste to-day, although companies to develop the power were chartered about seven years ago.

I incline to the view that a close study of these mill sites will show that there is no hope of finding a fair return on the investment for developing more than two out of these six sites, and that even in these two cases the economy, as compared with steam, is very doubtful. If this is true, then building the dam for New York near Merwinsville would cause no real damage.

Heavy damages would very likely be claimed, but from any estimate of possible revenue to be derived from sale of power, there must be deducted the necessary cost of development and interest on the cost of development and the interest that rolls up while waiting for customers for the whole. It is

slow work to build up a steady line for custom for long-distance electric power, and these locations are in a region where there is no local demand. It would beyond all question be more profitable for the owners of these charters and for the farmers who have not yet given options, to accept each his fair share in the diversion of, for all the nine possible power sites, let us say $500,000, than to go ahead and make further efforts to complete the control of mill sites, secure capital, build works and wait for customers for electric power.

Bull's Bridge Dam Site.

This is located on the Housatonic about one-half mile above confluence of the Ten Mile and nearly four miles upstream from proposed great dam. Has 780 square miles tributary watershed. Has run to waste for past twenty-five or thirty years. Was formerly a water-wheel here with 12 feet fall, which ran a blast furnace. Probably a fall of nearly 25 feet can be secured by canal of moderate length.

This is one of the sites secured by the New Milford Power Company, which is understood to have purchased the real estate formerly belonging to the iron works, but are understood to have not yet secured real estate controlling west end of dam, but probably have secured some flowage rights on west side upstream.

Kent Furnace Dam Site.

About five miles upstream from Bull's Bridge. Fall 8 feet. Drainage area 755 square miles. Has run to waste for five years. Formerly supplied power for an iron smelting furnace. Dam was blasted away five years ago by its owners—the iron company—to prevent danger from floods to the surrounding intervale lands, railroad, etc. The furnace company is understood to still own the adjacent real estate, dwellings and other buildings. All manufacture was abandoned here five years ago.

Swift Bridge Dam Site.

About seven miles upstream from Kent Furnace. Fall 15 feet. Seven hundred and twenty square miles of tributary watershed. Said to have been last used about thirty-five years ago for a grist-mill, with five runs of stone. Abandoned about thirty-five years ago and dam blasted away from fear of floods. Said to be owned by Edward Bierce.

Cornwall Bridge Dam Site.

One and one-half miles upstream from Swift Bridge. Dam 10 feet high—carried away twenty-five years ago in a flood. Tributary watershed about 700 square miles. Power said to have been formerly used for a saw-mill.

There is also a small country grist-mill on a tributary. Has only about one square mile of watershed, which has 12 feet fall on very small side stream entering at Cornwall Bridge.

There is also an abandoned power on another side stream entering at Cornwall Bridge, 10 feet fall. Formerly served an iron furnace.

Lest some future case be prejudiced by giving the final estimate of damages adopted in the estimate as most probable for each mill site, these details have been withheld. A lump sum for water diversion damages has been presented along with the detailed estimates of cost of reservoirs, which sum was arrived at after a study of many actual awards, and is believed to be liberal. To this a further reasonable sum has been added for probable expenses of litigation.

The foregoing data will enable any expert in this line to readily make up a check estimate.

TABLE

SUMMARY.—WATER POWER ON

That Would be Impaired by Diversion of Water for the Sup

Presented Subject to Revision.—The amount of fall and pondage and the extent to which the complete survey.

Serial Number. (1)	Locality. (2)	On What Stream Located. (3)	Area of Tributary Watershed, Square Miles. (4)	Reference to Description. For Ten Mile Division.	Reference to Description. For Housatonic Division. Page.	Average Net Fall, Approximate. Feet.	Present Condition. Developed, Undeveloped, Abandoned or now Proposed.
	Developed Mill Sites.						
1	Derby, Conn., Ousatonic Power Co.....	Housatonic R....	1,580	404	413	21.	⅗ developed...
2	Lanesville, Conn., Bridgeport Wood F. Co..	"	1,198	406	415	8.5	⅓ developed...
3	New Milford, Conn., Levi P. Giddings.....	"	1,183	408	417	9.0	⅓ developed...
	Proposed Power Sites.						
4	At Zoar, Conn., H. P. Co.....	"	1,532	410	419	50.	Undeveloped..
5	At Little York, H. P. Co.....	"	1,389	410	419	12.	" ..
6	At Southville, H. P. Co.....	"	1,219	410	419	25.	" ..
7	At New Bridge, N. M. P. Co.....	"	1,206	410	419	20.	" ..
8	At Gaylordsville, N. M. P. Co.....	"	998	410	419	15.	" ..
9	At Bull's Bridge, N. M. P. Co.....	"	795	...	420	25.	" ..
	Small Mills in Basin.						
10	South Dover grist mill.....	Ten Mile.....	200	398	...	9.	Intermittent...
11	Dover Plains, electric light, etc.....	"	134	398	...	8.	"
12	South Amenia grist mill.....	"	80	399	...	8.	" ...
13	Small saw and grist mill near Peet Hill	Small Brook.....	4	...	...	12.	" ...
14	Small cider mill near South Kent.....	"	3	...	...	10.	" ...
15	Very small intermittent power south of Cornwall Bridge.....	"	1	...	...	12.	" ...
	Abandoned Mill Sites.						
16	Kent Furnace	Housatonic.....	755	...	420	8.	Abandoned....
17	Swift Bridge.....	"	725	...	420	15.	" ....
18	Cornwall Bridge.....	"	700	...	421	15.	" ....
19	State Line	Ten Mile.....	215	...	398	15.?	" ....
20	Webatuck	"	201	...	398	8.	" ....
21	Brook near Cornwall Bridge.....	Small Brook.....	14	...	398	10.	" ....
22	Dover Furnace	Swamp R.....	32	...	398	12.	" ....
23	Brook near above	Small Brook.....	15	...	398	15.	" ....
24	Steel Works	.....	37	...	398	7.	" ....
25	Singpak	.....	1	...	398	6.	" ....
26	Wiminan Brook.....	.....	3	...	...	15.	" ....

Of the above, Nos. 13, 14, 15, 21, 22, 23, 24, 25, 26, are petty affairs,

No. 67.

HOUSATONIC AND TEN MILE RIVERS

ply of New York, by Dams at Merwinsville or Webatuck.

additional power may be developed are based on only a reconnaissance, and may be modified by

Total Horse-Power Available Before Diversion.				Approx. Amount of Power Taken Away from Maximum Development by Diversion of Ten Mile.			Approx. Amount of Power Taken Away, from Maximum Development by Diversion of Ten Mile and Upper Housatonic.		
Developed.			Undeveloped.		Developed 12-Hour Power.	Undeveloped 12-Hour Power.		Developed 12-Hour Power.	Undeveloped 12-Hour Power.
Greatest Amount of Power now Used, Reckoning all on 12-Hour Basis.	Now Available in Dryest Month of an Average Year.	Hours Per Day now Used.	Additional 12-Hour Power that could Probably be Developed for ⅔ the Year.	Proportion of Entire Watershed Diverted.	Net Horse-Power. × Months' Duration of Impairment.	Horse-Power. × Months' Duration of Impairment.	Proportion of Entire Watershed Diverted.	Net Horse-Power. × Months.	Net Horse-Power. × Months.
				Per cent.			Per cent.		
2,500	1,640	¾-24h. ¾-10h.	1,500	12.6	260 × 4	350 × 3	64.5	800 × 8	1,050 × 8
250 24h.	250 24h.	all 24h.	320 24h.	16.7	0 × 0	64 × 5	85.0	286 × 8	450 × 12
175	283	18 hrs.	425 24h.	17.8	0 × 0	142 × 5	93.3	70 × 12 24 hour.	344 × 12 24 hour.
0		0	9,000	13.6	0 × 0	1,225 × 12	66.5	0 × 0	6,000 × 12
0	0	0	2,000	14.4	0 × 0	288 × 12	73.3	0 × 0	Renders Development Impracticable if not so now.
0	0	0	3,500	16.4	0 × 0	574 × 12	83.5	0 × 0	
0	0	0	3,000	16.6	0 × 0	500 × 12	84.4	0 × 0	
0	0	0	2,000	20.8	0 × 0	400 × 12	100	0 × 0	
0	0	0	2,500	0	0 × 0	0 × 0	100	0 × 0	
70	60	10	40	Site flowed.	70 × 12	30 × 12	100	70 × 12	30 × 12
100	40	16	0	"	50 × 12	0 × 0	100	50 × 12	0 × 0
20	15	10	0	"	20 × 12	0 × 0	100	20 × 12	0 × 0
5?	0	?	0	"	5 × 1	0 × 0	.....	5 × 1?	0 × 0
5?	0	?	0	"	5 × 1	0 × 0	.....	5 × 1?	0 × 0
3?	0	?	0	"	3 × 1	0 × 0	.....	3 × 1?	0 × 0

0 The sum total of Developed Power that would be taken away from mills in Connecticut is seen by the table above and by the various estimates on pages 397 to 421 to amount to only :

0 For Diversion of Ten Mile River, about 260 Horse-power 4 months in the year.

0 For Diversion of Upper Housatonic, " 1,200 " 8 "

0 The impairment of opportunity to develop more power by additional turbines at sites already in use amounts to only :

0 For Diversion of Ten Mile River, about 520 Horse power 4 months in the year.

0 For Diversion of Upper Housatonic, " 1,500 " 12 "

0 At the proposed power sites, which have so far remained undeveloped, although the companies to develop them were chartered seven years ago, the impairment of opportunity to develop power would not

0 exceed :

0 For Diversion of the Ten Mile River, about 3,000 Horse-power 12 months per year.

0 For Diversion of the Upper Housatonic, " 20,000 " "

0 Considering the fair market value of the developed power in these localities and the great expenditure

0 required to make the undeveloped power of practical value, it is believed that the sums allowed in

0 Table 52, page 379, for damages to water rights are ample.

without possibility of any noteworthy value under present conditions.

Appendix No. 9.

YIELD OF ESOPUS CREEK.

The total volume of storage available in the reservoirs proposed by the Ramapo Company upon Esopus Creek above Olive Bridge, was estimated by Mr. George H. Rice, formerly Deputy Chief Engineer of the Croton Aqueduct Commission, and well qualified to judge of such matters, in his report to you dated August 29, 1899, page 4, at less than 12 billion gallons.

If this is the *most* storage that can be obtained, then it can be stated with reasonable certainty, that the safe yield of the Esopus Watershed for purposes of public water supply would not be more than 100 million gallons per day.

It is perhaps feasible to build the dams higher than proposed in the filed plans of the Ramapo Company and perhaps possible to choose better locations for some of them, and it may be thereby possible to obtain more storage and thus a larger yield, but at most not exceeding 150 million gallons per day. I have not had time or funds sufficient for investigating this difficult question of the dam sites exhaustively and cannot learn that the Ramapo Company or any other party has made such necessary investigations.

Under the *most liberal assumptions about rainfall and storage reservoir sites that can be made with reason,* the Esopus Creek alone, above Olive Bridge, can be made to yield only about the same quantity of water as the Ten Mile river, above the proposed Webatuck dam; while its dam sites are inferior to that at Webatuck and the chances are that the safe yield of the Esopus would be very much smaller than that of the Ten Mile river.

This yield of Esopus Creek, at Olive Bridge, is deduced as follows:

The drainage area of Esopus Creek, above Olive Bridge,
as found by tracing it out and planimetering it on the
best maps obtainable* is..... 242 square miles.

Mr. R. D. A. Parrott (see Scientific Amn. Sup., Sept. 4, 1886), in the original Esopus project for a water supply to New York, calls the Esopus drainage area 240 square miles.

The total capacity of the storage reservoirs proposed by the Ramapo Company, as found by Mr. Rice from first examining the filed locations in the County Clerk's office,† making notes therefrom and then going over the

* Map of 1899, compiled by Fisheries, Game and Forest Commission, and Colton's maps of Ulster, Greene and Delaware Counties, both on scale of 2 miles to 1 inch.

† These location maps filed by the Ramapo Company in the County Clerk's offices, in general do not state the elevation of the proposed flow lines or the heights of the dams and it is very difficult to obtain any definite information of their proposed plan of development from these maps. The filed maps are mere outlines, but from certain originals shown me by Mr. Nostrand, I infer they were all traced from topographical survey maps, giving full detail of contour and elevation. In answer to my questions about his dam sites Mr. Nostrand showed me only note book sketches of surface indications.

†† In fairness to Mr. Nostrand I desire to state that he very kindly offered to accompany me to the Esopus valley.

ground with aneroid and hand level and estimating the dimensions of the area to be flooded by the eye, was as follows. We quote Mr. Rice's report:

Olive Reservoir—"Would impound less than.....	2,000	million	gallons."
Boiceville—"Capacity is less than.....	2,075	"	"
Cold Brook—"Has a capacity of less than.....	3,000	"	"
Phœnicia— Shandaken— Big Indian— "These three reservoirs have a storing capacity of less than.	4,000	"	"
Total storage on main stream per Mr. Rice.....	11,000	"	"
Lake Hill—Capacity not stated by Mr. Rice.....			
"These seven available reservoirs as located by the filed plans of the Ramapo Company have a total capacity for storage of less than.....	12,000	"	"

In proportion to the drainage area, this storage found by Mr. Rice is

$$\frac{\text{12,000 million gallons}}{\text{242 square miles}} = \text{50 million gallons storage per square mile}$$

The total area of water surface exposed to evaporation would be not far from 5 or 6 square miles. Thus the ratio of water to land is 2 per cent.

Rainfall and Run-off.

There appear to be *neither rainfall records nor stream flow gaugings* available for this region. The Esopus Watershed is an ordinary "flashy" mountain watershed, *needing, with the same rainfall, more storage than a low-land stream to produce a uniform delivery*, and conversely affording a smaller safe yield per square mile in summer than a low-land stream, unless larger storage reservoirs are built. After considering the matters stated below, I have adopted for the Esopus the same yield per square mile as the Croton when provided with equal storage—granting that it may perhaps have a larger annual rainfall and a smaller evaporation but offsetting against these advantages the disadvantages common to flashy mountain streams.

The average declivity of the watershed is very high, causing a very rapid drainage of the surface and of the ground. The evidence available tends to show the rapid downpour of a large proportion of the total rainfall in sudden showers; a condition unfavorable to its soaking into the ground to reappear in springs with a steady yield in time of drought. A somewhat hasty view of the geological formation indicates that only the lower levels of the valleys are filled with deposits of porous drift, and that these deposits are narrow. The conditions appear very unfavorable for large, slow-moving ground water storage, suitable for equalizing the flow from February to October or for carrying over water from one year to another year.

On the steep rocky slopes there is less sponge-like storage in the soil to regulate the flow as it slowly drains out, than is found upon a low-land stream. Mr. Rice reports that at time of his examination the stream was so shrunken that one could cross it on the stones projecting from its bed, without wetting his feet. This implies an exceedingly low flow for a watershed of 240 square miles. High altitudes give small winter flows, and even on the Croton January is often a month of such small river flow that storage has to be drawn. Reservoirs on the Esopus would probably be later in replenishing than upon the Croton, and the extreme cold of winter would probably hold back the natural flow in larger degree.

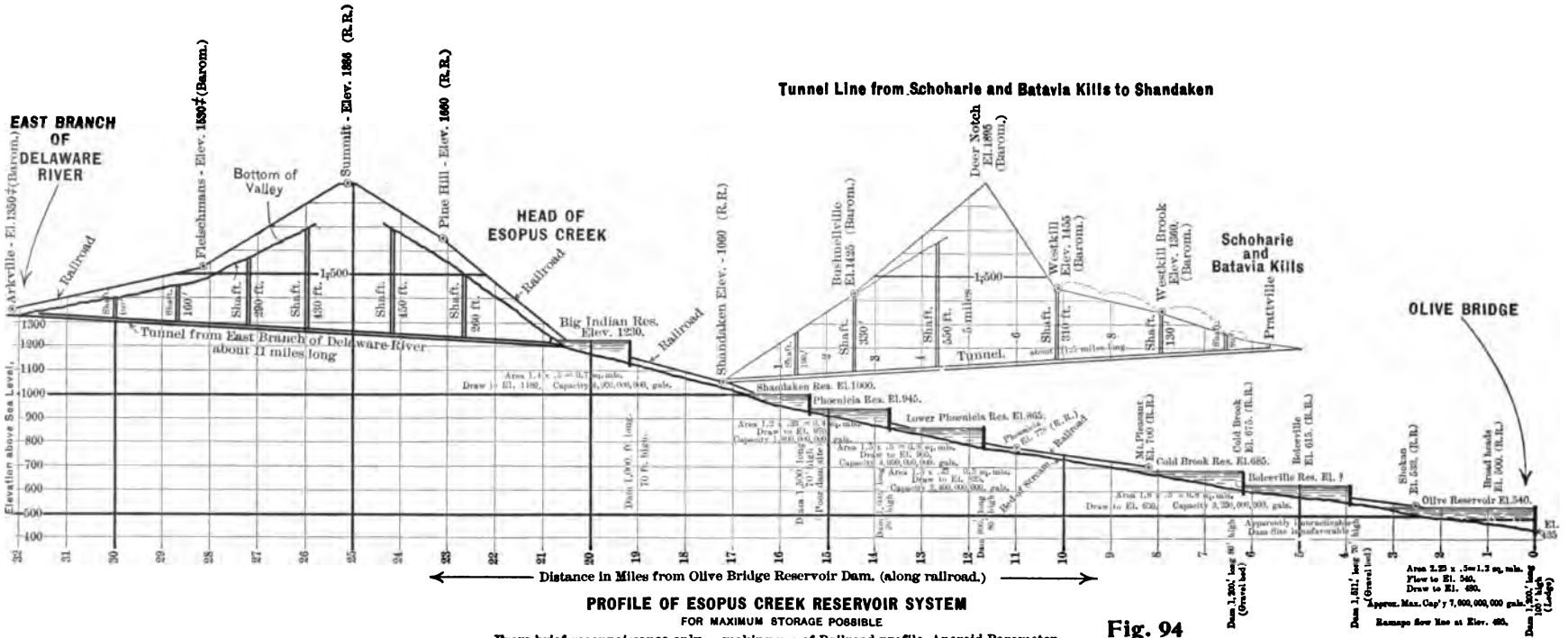
By the diagram of watershed yield derived from the Sudbury Records, and fully confirmed by our Croton Records, a watershed similar to the Sudbury, having only 50 million gallons of storage per square mile cannot be relied upon to yield *in years of extremely low rainfall,* more than 390,000 gallons per square mile of total surface per day (see diagram, Fig. 48). This, multiplied by the 242 square miles area of Esopus above Olive Bridge, gives, on both the Sudbury and on the Croton basis, a safe yield of only 94 million gallons per day.

Taking the most generous view that is reasonable and *taking the reservoir capacity from the filed locations* as estimated by Mr. Rice as correct, I believe we are not justified in reckoning the safe uniform delivery of the Esopus Creek alone at Olive Bridge, at more than 100 million gallons per 24 hours.

This yield of only 100 million gallons per day is so surprisingly small, from a source so confidently put forward by the Ramapo Company as the main feature in a proposed contract to supply 200 million gallons of mountain water per day, that a brief reconnaissance over the same ground was made by Mr. Ropes and subsequently by myself in order to learn if larger reservoirs could be built, and a larger safe yield thereby obtained. The results of this reconnaissance, looking for the maximum of storage, are in part set forth in the accompanying diagram, Fig. 94.

Larger Reservoirs on the Esopus.

From this reconnaissance and from certain maps very courteously shown me by Mr. Peter Elbert Nostrand, Engineer of the Ramapo Company, it appears that *if future borings and test pits prove ledge to exist in the bed of the stream and at its sides, within reasonable distance, then there may be* opportunity for developing more storage than estimated by Mr. Rice from the maps of reservoir location filed in the County Clerk's office by the Ramapo Company. Plainly, these location papers already filed need not limit the height of the flow line, for new papers could be filed.


Tunnel Line from Schoharie and Batavia Kills to Shandaken
EAST BRANCH OF DELAWARE RIVER
Arkville - El. 1350± (Baromr.)
Fleischmans - Elev. 1530± (Baromr.)
Summit - Elev. 1866 (R.R.)
Pine Hill - Elev. 1660 (R.R.)
Bottom of Valley
HEAD OF ESOPUS CREEK
Railroad
Tunnel from East Branch of Delaware River about 11 miles long
Big Indian Res. Elev. 1220.
Shandaken Elev. - 1060 (R.R.)
Shandaken Res. El. 1000.
Phoenicia Res. El. 945.
Lower Phoenicia Res. El. 865.
Bushnellville El. 1425 (Baromr.)
Deer Notch El. 1895 (Baromr.)
Westkill Elev. 1455 (Baromr.)
Westkill Brook Elev. 1260. (Baromr.)
Schoharie and Batavia Kills
Prativille
Tunnel.
5 miles
OLIVE BRIDGE
Mt. Pleasant El. 760 (R.R.)
Cold Brook El. 675. (R.R.)
Cold Brook Res. El. 685.
Belleville El. 615. (R.R.)
Belleville Res. El. ?
Shokan El. 528. (R.R.)
Broad Ripple El. 500. (R.R.)
Olive Reservoir El. 540.
Elevation above Sea Level.
Distance in Miles from Olive Bridge Reservoir Dam. (along railroad.)

PROFILE OF ESOPUS CREEK RESERVOIR SYSTEM

FOR MAXIMUM STORAGE POSSIBLE

From brief reconnaissance only; making use of Railroad profile, Aneroid Barometer, pacing and eye measurement. Subject to revision after instrumental survey.

Fig. 94

While such larger reservoirs may be possible and will be estimated upon in the effort to do full justice to the Esopus project, I was, myself, very unfavorably impressed with the apparent lack of opportunity for large storage. The Olive Reservoir has a magnificent exposure of bed rock but is the only one on the main river where there is ledge visible at the proposed dam site. At the other reservoirs (excepting the Lake Hill site on a side stream) the indications are very unfavorable for safe foundation for a high dam at any reasonable expense. The upper valley was apparently originally much deeper than now and filled with drift, which has been but partially eroded. The narrow places in the valley selected for dam sites are caused apparently by porous drift, and not by spurs of ledge. The terraces or benches of drift from 15 to 30 feet above the bed of the river cut off a good deal from the reservoir capacity that would naturally come from so high a dam, and with a low dam such as might be safe without going to bed rock, the volume of storage would be far too small.

Until some one puts down test pits and proves the presence of ledge within reach and that conditions are safe for high dams, I am personally disposed to regard 100 millions as the utmost safe capacity of the Esopus with the utmost safe storage.

Atwood Reservoir—It is positively stated in the Ramapo Company's proposition to the city that no water was to be taken from a lower elevation than 420 feet above sea level, and this restriction cuts out the use of the Atwood (about elev. 180) and several other reservoirs on which locations had been filed.

Ramapo Company's Reservoir Maps.

Olive Reservoir—From certain survey plottings on contour maps, scale 200 feet to 1 inch, shown me by Mr. Nostrand, and which bore all appearances of being carefully made maps, suitable for water supply studies, I noted roughly that the contour lines of the map for the Olive Reservoir, with flow line at 495, bottom of valley at dam site being about elevation 435, would give a total storage in Olive Reservoir of, roughly, 4 billion gallons, and that the map indicates the contour of the valley favorable for a flow line at elevation of 540, by flowing out the railroad and the adjoining Village of Shokan, in which case, according to said maps of the Ramapo Company, the storage might reach the 7 billion gallons, stated by Mr. Dalton (see "City Record," August 23, p. 5271) instead of the 2 billion gallons found by Mr. Rice for the smaller and more reasonable dam and the less expensive flowage; but it would from my reconnaissance appear to require extensive borings in the north bank to prove that a dam over 100 feet high was safe without very expensive wing walls.

Cold Brook, etc.—I was not shown the Ramapo Company's plans of the Cold Brook, Boiceville, Phoenicia, Shandaken and Big Indian Reservoirs, which Mr. Rice's reconnaissance gave in all as containing 9 billion gallons *per filed plans*. My own reconnaissance gives the probable *maximum development, regardless of poor or doubtful quality of dam sites*, at about 15 billion gallons.

Mr. Dalton's statement in his communication to the Board of Estimate (see "City Record," August 23, 1899, p. 5271) that "Coal Brook Reservoir" can contain 15 billion gallons, may be thus explained if taken to mean the maximum possible development of the series of reservoir sites above Cold Brook, on the Esopus, regardless of doubtful foundations.

Lake Hill—At the Lake Hill Reservoir the map shown me by Mr. Nostrand bore all ordinary appearances of being reliable, and the drawings indicated a reservoir site of unusual excellence and economy, and showed a higher water supply value than found by Mr. Rice. The Kaaterskill

sheet of the U. S. Geological Survey also shows this Lake Hill Reservoir site, and tends to confirm the accuracy of the map shown me by Mr. Nostrand.

The drainage area naturally tributary to Lake Hill, as traced out on the Forest Preserve Map and U. S. Geological Survey Map is..... 19.5 square miles.

Mr. Nostrand told me that he had planned for flow line at 1,050 ; bottom of valley being at elevation 990 above sea, and dam being only 125 feet wide at bottom and 400 feet wide at top—"solid rock all around dam site."

By the Ramapo surveys the Lake Hill Reservoir Dam can be cheaply raised to any reasonable height required ; this is confirmed by the Geological Map, but was not investigated on the ground by Mr. Ropes and myself. If developed to yield in driest years 800,000 gallons per square mile per day or 16 million gallons in all, a storage of 3.5 billion gallons would be needed.

It therefore appears that if higher dams than shown on the original locations can be safely built, which is doubtful, and thus flowing out the railroad and many houses, a total storage of $7 + 15 + 3.5 = 25.5$ billion gallons of storage could be obtained, which is double the amount of storage found by Mr. Rice from hasty reconnaissance based on filed plans, thus getting a total as large as the 7 plus the 15 billion gallons mentioned by Mr. Dalton ; but, as already stated, the surface indications are very forbidding for dams of such great height.

YIELD OF ESOPUS WITH LARGEST POSSIBLE STORAGE.

(Disregarding for the present the question of leaky dam sites.)

To compute the yield that these larger reservoirs would maintain, the watershed yield should be computed in two parts. First, all of the main stream, and second, the Lake Hill district, because these will have different rates of yield.

The 6 or 7 reservoirs on the main river lying as they do in close proximity and with the downstream reservoirs catching the waste from the upstream reservoirs can be so managed as to yield substantially the same as if the whole of this 22 billion gallons of storage was in one basin at the outlet.

$$\frac{\text{22 billion gallons storage}}{\text{222 square miles}} = \text{100 million gallons storage per sq. mile of land.}$$

This, on the Croton and Sudbury basis, per Diagram No. 48, with 3 per cent. of water surfaces, should yield a safe, daily flow of 575,000 gallons per square mile, which, for 222 square miles, amounts to $222 \times 575{,}000$.....	128	million	gallons	per	day.
Adding from the Lake Hill territory.....	16	"	"	"	"
Esopus total yield.....	144	"	"	"	"

We thus find that the greatest safe yield from the Esopus above Olive Bridge, with maximum development of storage, would not, in round numbers, exceed 150 million gallons per day.

To gain this large volume of storage would drown out a large number of houses and cover the larger part of the good land in the entire valley and throw the railroad much of the way into the steep side hill and call for a large number of very expensive dams. *The conditions for dam building are not favorable* except at Olive Bridge and Lake Hill. We may therefore consider the greatest possible yield of the Esopus, according to the amount of

storage that can be reasonably developed, somewhere between 75 and 150 million gallons per day.

No maps have been published giving data from which the storage-reservoir possibilities of the adjacent watersheds can be accurately determined. The only way to learn them definitely is to put a survey party in the field equipped for stadia surveys and levels, and to spend money liberally for test pits and borings seeking for ledge at the dam sites, and to have each locality carefully studied by some competent geologist, as I have already had done for the Webatuck and Merwinsville sites on the Ten Mile and the Housatonic.

It is a public misfortune that the State of New York has not in years past co-operated more heartily with the U. S. Geological Survey in the preparation of topographical maps, so that these excellent maps would now be available over all this territory, in which water supply can be sought; and no better expenditure toward an exhaustive study of these water supply problems can be made, than to hasten the completion of these maps throughout this region. In the neighboring States of New Jersey, Massachusetts and Connecticut, a complete topographical map, giving contours of elevation, is available throughout the entire area of the State.

WATERSHEDS ADJACENT TO THE ESOPUS.

The Parrott Plan—The starting point, in public recommendation of the Esopus as a water supply for New York, is found, so far as I can learn, in a paper published by a Mr. R. D. A. Parrott in the Scientific American Supplement on September 4, 1886, written against the Quaker Bridge Dam and made up largely of froth. In the last few paragraphs of his article an earnest recommendation was made of a water supply to be obtained from the Catskill Mountains, and a map was presented, of which I attach a copy (see Fig. 95). The Ramapo Company's locations of dams, drainage areas, points of diversion and direction of tunnels, so far as made public, follow this old map to a very remarkable extent.

In Mr. Parrott's article and map it is shown how, when the Esopus has been exhausted, the following additions can be made:

1st. To the north, the head waters of Schoharie Creek from 110 square miles can be brought to Esopus by a tunnel eight miles long.

2d. Still further north, Batavia Kill, with 50 square miles of drainage, is to be brought into the Schoharie stream above mentioned, by a tunnel 3 miles long, and after following the stream about 3 miles, enter and follow the 8-mile tunnel to Esopus first mentioned.

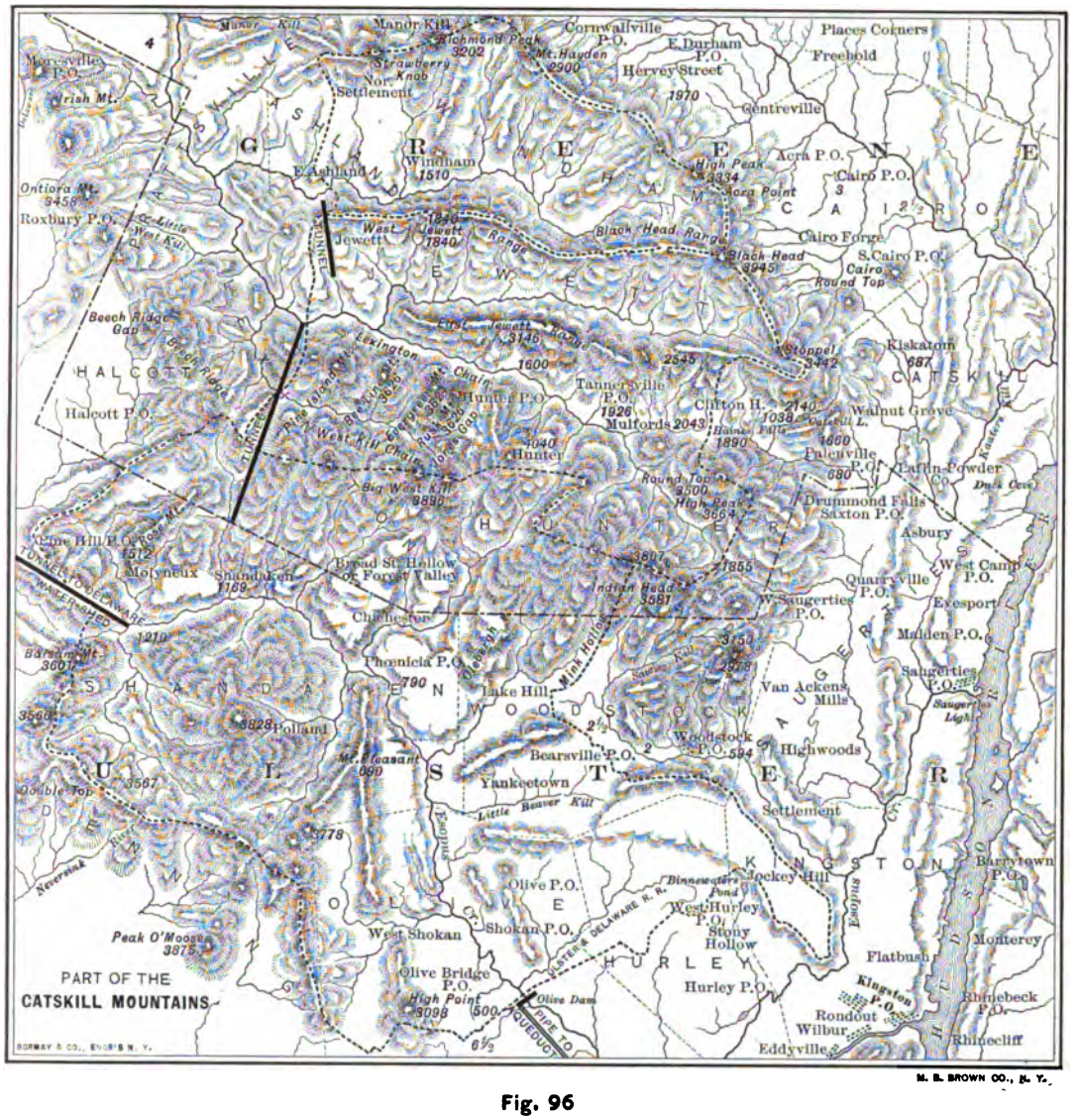


Fig. 96

A copy of the Parrott Map of proposed Tunnels and Aqueduct for Supplying New York City with Water from Esopus Creek and neighboring Water Sheds. (See *Scientific American* Supplement, September 4, 1886.)

3d. Next starting on a new line more to the westward, a tunnel from Big Indian on the Esopus, at elevation 1,210 feet, running 8 miles northwest to Deane's Corners at elevation 1,344 feet, would, it is said, drain 130 square miles and bring in Dry Brook, Bush Kill, and the head waters of the Delaware river.

Mr. Parrott says the total drainage area of these is

Esopus	240	square	miles.
Schoharie Creek	110	"	"
Batavia Kill	50	"	"
Delaware, Bush Kill, Dry Brook	130	"	"
Total	530	"	"

He goes no further with definite tunnel propositions, but says that "no doubt both the Delaware and the Neversink could be made to contribute in addition the flow from a thousand square miles," and further, that with tributary watersheds brought in by short tunnels as above described, "the time for resorting to storage will be long postponed."

All these matters of the Parrott proposition are so nebulous that I will only take time to note that his tunnel to Schoharie Creek, 8 miles long, as shown on the map, runs deep under a high mountain with very little hope for more than one practicable shaft and this shaft (on the West Kill) about 400 feet deep. In fairness it must be said that this map was intended to be illustrative rather than exact and that a better line can be found under the neighboring valley.

An arch-lined tunnel of the smallest practicable size and length between headings would cost not less than $150,000 per mile and would more probably cost $175,000. This tunnel, without the lining, would cost not less than $80,000 per mile and more probably $90,000. If the tunnel is arched and lined for half its length and the rock found sound enough to warrant leaving the other half unlined, the cost will average not less than $110,000 per mile.

8 miles of 6 foot tunnel, at $110,000	$880,000
The shafts will cost not less than $100 per foot, or at least	80,000
	$960,000
Add, as ordinarily, for supervision and contingencies, 15 per cent	145,000
Probable cost of Schoharie Tunnel at least	$1,105,000
The Batavia Kill tunnel, 3 miles long, could hardly cost less than	300,000
A total for these two tunnels by the Parrott plan of at least	1,405,000
And possibly with more arching, particularly if given a less steep slope in order to accord more nearly with ordinary practice, the cost might rise to	2,000,000

The Ramapo Company's tunnel location as indicated upon a map filed in the County Clerk's office at Kingston runs from the mouth of the Smith Bush Kill, north 1 degree 15 minutes west, taking advantage of the clove or valley.

More Favorable Plans Possible for Schoharie and Batavia.

In justice to the possibilities of supplementing the waters of the Esopus by water from Schoharie Creek, we note from our hasty reconnaissance a more favorable tunnel line, although longer; and the opportunity to tap the Schoharie and Batavia Creeks at a lower elevation where the combined watershed is 75 square miles larger, and where the yield of the Schoharie and Batavia can probably be made 50 per cent. more.

This new line would average about 15 degrees farther to the westward to tap the Schoharie Creek at Prattville, 5 miles further downstream than Lexington. This Prattville tunnel line would contain three slight angles in order to secure more favorable locations for shafts. It is entirely possible that accurate levels and more complete study would show it possible to divert Schoharie Creek at a point still further down stream where the yield would be still larger.

Other matters, seemingly of more importance in the brief time at my disposal, have prevented the full development of this reconnaissance, but from the maps and two or three brief views of a part of the country it appears feasible to divert the drainage from about 235 square miles of watershed on the Schoharie Creek and the Batavia Kill combined; or substantially the same drainage area as that above Olive Bridge, and that the storage possibilities in quantity and cost are as good or better on the Schoharie than on the Esopus. In other words, the Schoharie Creek and the Batavia Kill will together yield just about the same total quantity of water as the Esopus, and that this will be anywhere from 75 to 150 million gallons according to the quality of dam sites, the imperviousness of the drift deposits, the height of dams and the allowable expediture for storage.

So the Esopus, the Schoharie and the Batavia combined would safely yield in dryest years, anywhere from 150 to 300 million gallons per day, according to the storage capacity constructed.

Eleven miles of tunnel from Shandaken to Prattville, in comparison with the 30 miles of tunnel on the New Croton Aqueduct, may not appear large, but it makes a great difference whether, like the Hoosac, the Mount Cenis or the St. Gothard, it must be driven mainly from portals miles away from the heading, or whether, like the New Croton or those proposed between Sodom and Park Hill in another part of this report, there can be short shafts as near together as desired, and many gangs worked simultaneously.

The 8-mile tunnel proposed by Parrott and apparently followed by the Ramapo Company, westward from Esopus Creek at Big Indian, reaching out for 130 square miles of the watershed at the head of the Delaware river, appears to be about 11 miles long by the latest maps, instead of 8 as stated by Parrott, and from the maps and general appearances not an advantageous proposition, but I have not had opportunity for any thorough study of it. It appears easy to obtain a better line, having frequent shafts as roughly indicated in Fig. 94, than the shorter line on the Parrott map which indicates the tunnel running for fully 5 miles lengthwise of a high mountain with no possibility of a shaft.

Tunnel from Esopus to Headwaters of the Delaware.

This part of the Parrott plan has been followed to some extent in the Ramapo Company's plans, judging from certain of the locations filed. Certain questions regarding the right to divert an interstate river might come up in this connection the same as for the Ten Mile river; but probably in much less serious form, because of the small proportionate part of the entire flow of the Delaware, that would be diverted.

It appears that there is a practicable tunnel line by following nearly the line of the railroad, thus obtaining shafts at reasonable distances apart with none deeper than 450 feet, but this tunnel line would nevertheless be very expensive and a very slow one to construct.

I have not had time to investigate the possible dam and reservoir sites on these streams.

The later maps indicate that 167 square miles of watershed can be obtained, instead of the 130 stated by Mr. Parrott, and if we may assume that there is a possibility of developing as much storage as found by Mr. Rice per square mile of the Esopus Watershed, it may be possible to obtain about 63 million gallons per day in extreme drought. This corresponds to 380,000 gallons per square mile of gross area, and requiring, according to Diagram No. 48, a total available storage of 50 million gallons per square mile, or about 8.3 billion gallons of available storage for the entire 167 square miles.

Although with the steep hydraulic gradient attainable, the whole safe yield of this 167 square miles could be taken through an unlined tunnel as small as could be driven, one could not hope to carry a tunnel for these great distances so small as the standard short mining tunnels of the Rocky Mountain region, 4 feet wide and 6 feet high, and it would be hopeless to expect to carry the smallest and cheapest possible tunnel through to these small high tributaries of the Delaware for less than a round million dollars, and a cost of a million and a half dollars is much more probable for the tunnel alone.

To this must be added the cost of the impounding reservoirs and the cost of the water rights.

I regret not to have had the opportunity, prior to the time set for submitting this report, to examine more fully into this matter of diverting the head waters of these tributaries of the Delaware into the Esopus, but from the brief consideration outlined above it does not appear an attractive or economical proposition.

COST OF ESOPUS AQUEDUCT LINE.

This reconnaissance and estimate was begun with a view to comparing the economy of water from the Esopus with that from other possible sources and with the intention of including approximate estimates of cost of dams and reservoirs sites and water diversion damages.

The various problems were found to be so complicated and the data on safety of dam sites so very uncertain that to give a detailed estimate required more field work than the time afforded.

The shortest distance from Olive Bridge to the city limits in an air line is about 74 miles. The shortest practical line with steel pipe and tunnels appears to be 78 miles, and the more economical route using cut and cover aqueduct largely 86 miles, while from a filtration-plant pumping-station at Poughkeepsie the distance is 61.5 miles, and from East Branch Reservoir over the route proposed for bringing in the Ten Mile and Housatonic water the distance is only about 36 miles. The Ten Mile or Housatonic water would have precisely the same elevation as proposed for the Esopus water at city limits, namely, 300 feet above sea level, and the upstream end of the aqueduct be 42 miles nearer, with even more safety and freedom from liability to rupture in a long conduit than this saving in distance implies, because of it having no such very difficult and unprecedented structures as those required for crossing the Hudson river.

The project of the Ramapo Company, as I gather by conversation from Mr. Nostrand, was to run two steel pipes, each 8 feet 3 inches diameter, under a slope of about 2 feet per mile, all the way to New York, with the exception of some short lengths of tunnel. It was proposed to cross the Hudson in a deep tunnel probably near Highland. These pipes were proportioned, as Mr. Nostrand tells me, with a coefficient of flow derived from a value of $n = 0.16$ in the Kutter formula, and I concur with the result of his estimate that pipes of this size with lap courses and projecting rivet heads, and after considerable fouling, could be prudently relied upon to convey 200 million gallons per day with a slope of 2.0 feet per mile.

My studies of this question give promise of a cheaper and more durable form of conduit for a part of the line than that proposed by Mr. Nostrand, by availing of the possibility for constructing a considerable portion in cut and cover masonry, instead of steel pipe.

To put the Esopus project in its best possible light I have prepared the following estimate, which is probably on as favorable a line for cost as that proposed by Mr. Nostrand, Chief Engineer of the Ramapo Water Company.

Unfortunately the excellent topographical maps of the United States Geological Survey have not yet been extended over much of this region west of the Hudson. The conduit location which appears to be the cheapest is one starting from Olive Bridge and following the trend of the high ground at an elevation appropriate for cut and cover aqueduct, to the greatest possible extent and without too great deviation from the general direction of the proposed Park Hill Reservoir site near the New York City limits.

A hasty and very incomplete examination of portions of the country through which this line will run seemed to show that such a line would follow the bluffs and slopes on the south side of

the Esopus Creek Valley for some miles, then cross the Rondout Creek Valley in the vicinity of High Falls by steel pipe siphon. Thence around the north end of the Shawangunk Mountains, probably with a tunnel of moderate length; and, after taking advantage of the slopes on the east side and near the base of these mountains, so far as can be done reasonably for cut and cover construction, cross the Wallkill Valley near Libertyville with a steel pipe line and occupy ground of suitable height on the ridge easterly from Wallkill river. Thence running southward along this ridge, mostly with cut and cover aqueduct, until the conduit crosses the divide, at some point southerly from Orange Lake, with a tunnel perhaps 2 or 3 miles long and one easily worked with short shafts. The valley of the Moodna Creek would be crossed by steel pipe siphon at suitable point between Mountainville and Orr's Mills. A tunnel about 4 miles long would be required through the north portion of the Highlands (under Black Rock Hill and southeasterly from there). The chances for a desirable number of shafts along this tunnel are not very promising and one drift may have to be at least 2 miles long. Thence crossing the Hudson at the best point between Highland Falls and Fort Montgomery, the line continues through the Highlands on the east side of the Hudson river, passing under the hills and crossing the valleys with alternating short stretches of tunnel, pipe line and cut and cover to a point in the Croton Watershed 4½ miles east of Peekskill, where an admirable site for a 3 to 5 billion gallon equalizing reservoir at grade 370 full, 340 drawn, can be utilized by building a dam at Holman's Mills.

Thence the water could be carried about half the way by steel pipe and half by cut and cover masonry aqueduct, down to the city limits.

The total distance by this, apparently the most economical line from Olive Bridge to the city limits, is 86 miles. By a study of the topographical maps of the U. S. Geological Survey which cover the southerly portion of this line, supplemented by an incomplete reconnaissance in this region for other purposes, it appears reasonable to expect that the cost of such a conduit line, nominally of 250 million gallons per day capacity, but expected to serve for only 200 million gallons, after making the allowances that have been made in other parts of this report for impairment and surplus, would be *at least* $22,000,000. The estimate for this follows upon the next page. The proposed aqueduct starts with a hydraulic grade at Olive Bridge, when reservoir is drawn down to elevation 485 feet above sea, and ends at the New York City limits 2 miles north of Jerome Park, at a hydraulic grade of 300 feet above sea.

A conduit (or conduits) of 400 million gallons actual average daily capacity, or 500 million gallons nominal capacity per day, when clean, built along the same line, would probably about cost $35,000,000, but there is no hope of sufficient water in this Catskill region being diverted above grade 420 to fill conduits of this size.

TABLE NO. 68.

Estimated Cost of Conduits, Olive Bridge Reservoir to City Limits (Woodlawn), Distance, 86.0 Miles (Say, 25 Miles of Steel Pipe, 50.5 Miles of Cut and Cover, and 9 Miles of Tunnel).

Assumption—Hydraulic Grades, 485 at inlet and 300 at outlet.

50.5 m. C. & C. Aq. at 1.6′ p. mile	80 8 ft.
9.0 miles tunnel at 2.0′ p. mile	18.0 "
25.0 miles pipe at 3.0′ p. mile	75.0 "
Hudson River Tunnel 4.0′ p. mile	6.0 "
Minor losses	5.2 "
Total head	185.0 ft.

Daily average supply 800 million gallons.

Capacity (clean) 250 million gallons.

Average diameter of cut and cover aqueduct 10.5 ft.

Note—Holman's Reservoir can be drawn to grade 350± by this project, giving an excellent equalizing reservoir.	Cost per unit.	Total cost.
50.5 m. C. & C. Aqueduct	$142,000	$7,171,000
4.5 m. Arch Lined Tunnel	305,000	1,377,000
4.5 m. Side Lined Tunnel	248,000	1,116,000
1.5 m. Hudson River Tunnel, excavation, steel lining, concrete, etc., complete.	.....	900,000
25.0 miles $\frac{7}{8}$ inch pipe	Diam. 8 ft. 3 in. 153,000	3,825,000
Present value of second line of pipe deferred 6½ years, discounted at 3% per annum	.....	3,156,000
600 lineal feet shaft excavation	100	60,000
Bridges, Culverts, Blow-offs, etc., on 77.0 miles	15,000	1,155,000
23 Siphon Chambers	10,500	242,000
Right of way, 50.5 miles cut and cover aqueduct	3,000	152,000
Right of way, 9.0 miles tunnel	2,500	23,000
Right of way, 25 miles pipe	60 ft. wide. 1,800	45,000
	.....	$19,222,000
Add 15% for contingencies and supervision, etc.	.....	2,883,000
Total cost of conduit line	.....	$22,105,000
The estimated additional cost for terminal and equalizing reservoirs, which apparently have not been contemplated in the Esopus project as proposed by the Ramapo Water Company, is as follows:		
Equalizing reservoir at Holman's Mill—Roughly, say	.....	1,189,000
Terminal reservoir at Park Hill (North Basin only)	.....	3,696,000

Storage Requirements for Safe Yield of 200 Million Gallons Per Day "from the Esopus and Connecting Watersheds."

The size and therefore the cost of the required storage reservoirs will vary according to the magnitude of the safe yield that the given stream must provide. The two hundred million gallons proposed can not be had from the Esopus alone, however high its dams are built and however great its storage. The cost of the additional water beyond what the Esopus can furnish depends on its source.

In the proposition of the Ramapo Water Company (see *City Record*, p. 5270, August 23, 1899), it was proposed to supply on "each and every day" 200 million gallons of water "from the Esopus and its tributary and connecting watersheds in the State of New York"; and that the water "shall not be drawn from any stream or lake the surface of which is at an elevation of less than 420 feet above mean tide level."

For the safe delivery of this 200 million gallons per day from Esopus, Schoharie and Batavia creeks, in years of extreme drought, with same rainfall, evaporation and steadiness of ground yield as on the Croton, a storage capacity of at least 30 billion gallons will be required, and there is, up to date, much doubt if this capacity can be safely obtained at any reasonable cost.

We have seen, page 426, that with the reservoir locations as filed, and according to the approximate storage capacity estimated by Mr. G. S. Rice, the safe yield of the Esopus creek itself would not exceed..... 94 million gallons per day.

While by developing storage reservoirs to the utmost by dams of uncommon height considering the poor prospect for ledge at most of these dam sites, the safe yield of the Esopus could possibly be developed up to.....150 million gallons per day.

If the Esopus were developed with no more reservoir capacity than is common in water supply work on such streams, the safe yield would probably not exceed .....from 75 to 100 million gallons per day.

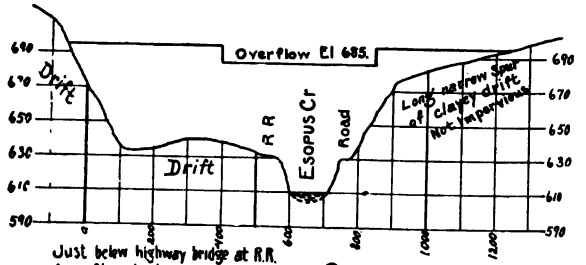
—leaving to be obtained from "connecting watersheds" .....125 to 100 million gallons per day.

Presumably the Schoharie would be sought first, since that is nearest and largest and involves no interstate complications like the Delaware and would yield softer water than the Catskill creek.

To put the matter in a favorable light as is reasonable, assume that ledge at the upper dam sites is not beyond reach and that there is ledge within reach beneath the side benches of drift at the site of the wing walls of the dams or that the drift is safely impervious.

Assume the Schoharie creek diverted below the Batavia Kill, at Prattville, where drainage area is 180 square miles. Then if same proportion of storage is available on each stream, it will require upon the combined 240 + 180=420 square miles of watershed, a total storage capacity of 30 billion gallons. This is 72 million gallons per square mile of gross area which per the Croton and Sudbury experience as set forth in Diagram No. 48 would develop a safe yield in extreme drought of 476,000 gallons per square mile per day, or 200 million gallons per day from the 420 square miles.

We have already discussed the reason for more storage being required per square mile, to maintain a given safe flow on these mountain streams, than in a valley like the Croton or Sud-



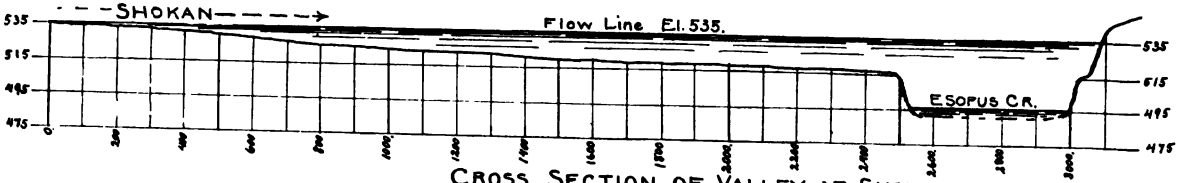
COLD BROOK DAM
ON ESOPUS CREEK

Fig. 96.

CROSS SECTIONS AT DAM SITES ON ESOPUS CREEK

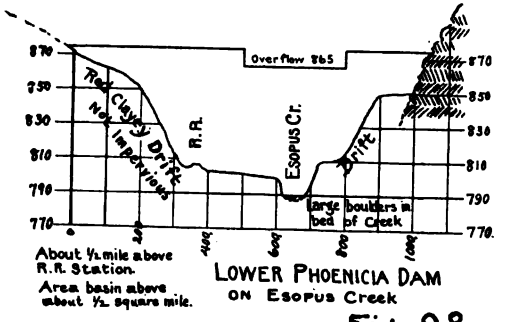
From brief reconnaissance only, aided by aneroid barometer, pacing and eye measurement. Subject to revision after instrumental surveys.

Sections made by Horace Ropes, C.E.



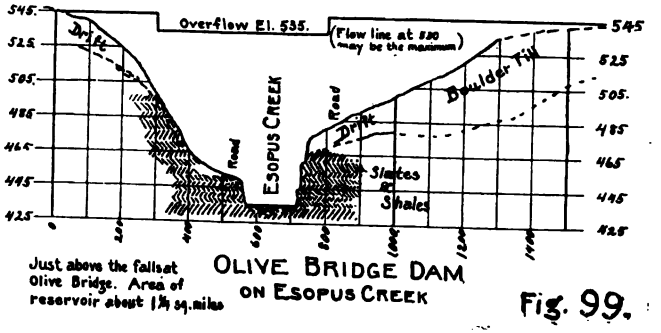
CROSS SECTION OF VALLEY AT SHOKAN
on ESOPUS CREEK

Fig. 97.



LOWER PHOENICIA DAM
ON ESOPUS CREEK

Fig. 98.



OLIVE BRIDGE DAM
ON ESOPUS CREEK

Fig. 99.

bury; and it is probable that more than 30 billion gallons capacity in storage reservoirs must be provided to make sure of a delivery of 200 million gallons per day from the Esopus, Schoharie and Batavia creeks.

As against the possibility that so large a storage as 30 billion gallons can not be safely and economically secured above Olive Bridge and above Prattville, we have the possibility of finding it possible to go below Prattville and obtain somewhat more watershed.

Probable Cost of Storage on Esopus Creek.

This storage must cost more per million gallons than storage has cost upon the Croton, because the dams are more difficult than those already completed on the Croton tributaries and also because these mountain valleys are steep and narrow, with terraces or benches of drift near the bottom, and will hold less storage for a given size of dam. While in general it is difficult to measure the cost of a proposed reservoir by multiplying its capacity by an assumed cost per million gallons, we can draw such comparisons with other reservoirs as will serve for a fair approximation to check these rough Esopus estimates in advance of detailed instrumental surveys.

In the broad valleys of the Croton Watershed the total cost of providing storage is understood to have averaged about $200 per million gallons of reservoir capacity (see also Brooklyn Report of 1896, p. 16).

From such information on individual reservoirs as I have been able to collect this figure apparently refers to the earlier and smaller basins. Some of the enormous basins constructed in recent years, having each more than a square mile of water surface, formed by short dams across the outlet of broad valleys, appear to have cost considerably less, but the available published figures for these do not include cost of land, legal expenses, supervision during construction and preliminary surveys. In these Croton reservoirs railroad locations did not have to be changed, comparatively few house sites were flooded, and the water-power damages were small or absent. The per-gallon basis is an uncertain one on which to estimate cost of storage reservoirs, for the cost will evidently vary with type of dam and depth of foundations; but, after examining the various reservoir sites and dam sites on the Esopus, and from our reconnaissance preparing the rough sketches shown in Figures 96 to 99, and making rough comparisons with what other water-works dams of similar length have cost, and reckoning on work of only the most superficial character in removing stumps and grubbing the reservoir beds and margins, considering the absence of visible bed rock at most of the dam sites, the narrowness of the valleys and the steep declivity of the stream, the necessity for railroad relocation, the villages flooded and the mill sites impaired, it appears that the total cost of the Esopus storage

will probably average not far from $300 per million gallons of capacity, and from the brief inspection made we will assume that the storage on Schoharie creek and Batavia Kill will, as a whole, cost the same.

30 billion gallons storage capacity at $300 =..... $9,000,000.

It appears probable that the Schoharie and Batavia tunnels to the Esopus, of a total length of 11.5 miles, could be built for somewhere about $2,000,000

We thus have for a rough approximation of the probable cost of all the lands, rights and structures necessary for collection, storage and transportation of a safe yield of 200 million gallons per day "from the Esopus and its tributary and connecting watersheds":

Lands, rights and storage reservoirs.....	$9,000,000
Tunnel from Batavia and Schoharie creeks to Esopus.....	2,000,000
Aqueduct, Olive Bridge to New York City limits.....	22,105,000
Probable total cost of rights, lands and structures for the Ramapo Company's proposition.....	$33,105,000

—not including the terminal reservoir at Park Hill and the equalizing reservoir at Holman's Mill, since none were provided for in the Ramapo Water Company's proposition.

Smallest Cost for the Esopus Project.

In the foregoing estimate of Esopus cost, the effort has been to take the reasonable view and to provide for structures of the character common in good municipal work.

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*Separate estimates were made for each site but data were so rough that it is not worth while to transcribe the computations.

Transcribing briefly a few of our notes regarding these dam sites.

At Olive Bridge it was noted that that site seemed better adapted for a dam of 80 feet than higher. The slate strata in bed of river remarkably level. Out crop shows rotton stone in places. A high dam would completely destroy Villages of Shokan, Olive Bridge, Brodhead and Shokan; Containing say 60 homes and perhaps 400 people. The series of high dams for full development of watershed would require rebuilding substantially entire length of railroad from one mile east of Brodhead to one mile west of Big Indian, a total distance of 21 miles. At Boiceville no location that suggested a good dam site was found. At Cold Brook no indications that bed rock was within economic reach. Narrowing of valley is caused by a spur of drift the imperviousness of which under high head of water is very doubtful.

Above Cold Brook there is a good deal of red clay mixed in with the drift, which washes badly in spring floods and in heavy showers, giving a deep chocolate color to the stream, which subsides very slowly indeed as shown by pools of long standing. Data as to wells indicated a modified drift formation in many places—"Water bearing" gravel.

A large part of the cost of a proper water supply plant comes from the precautions against the contingency of interruption to the supply by accident, and in providing against the remote contingency of shortage of water in those cycles of low rainfall which occur at intervals of ten or twenty years.

A railroad bridge must not merely be strong enough 99 times out of 100, but must be sufficient every time, and so the universal practice is to put in about 3 or 5 times as much metal as would be barely sufficient to carry the greatest load that can be foreseen.

Similarly a water supply system must be of ample capacity not to break down, or be left with empty reservoirs during the most severe drought of 32 or 50 years. These cycles of successive years of low rainfall come so seldom that the public at large does not appreciate their danger.

By letting the public take its chances, while the water-supply contractor runs for luck, and by using cheaper types of conduit, the cost above estimated could probably be cut down fully five million dollars, and it is nine chances out of ten that there would be water enough and that the conduit would not burst.

It is also possible that with liberal storage and with excellent structures, the conditions may be more favorable than I have assumed, and that perhaps $2,500,000 can be saved over the estimate presented on the preceding page.

Largest Reasonable Cost for the Esopus Project.

Looking at the problem from the opposite point of view, to see wherein the margin of uncertainty due to incomplete surveys may possibly make our estimate of cost too small, we obtain as an outside estimate the figures summarized in the right-hand column of Table No. 7, page 103.

Notwithstanding the lack of completeness in our surveys, the cost can hardly fall outside the maximum and minimum values presented in Table No. 7.

Appendix No. 10.

SUPPLY OF FILTERED WATER FROM HUDSON RIVER ABOVE POUGHKEEPSIE.

This is one of the two or three projects, which, while not so promising as the Ten Mile and Housatonic gravity supplies, is worthy of further study in detail by some expert in matters of filtration, and with a detailed study of the pump location, the filter location and the aqueduct line.

The great obstacle to providing New York with filtered Hudson river water is the 60-mile aqueduct required to get the intake far enough upstream to be sure that the water will never be brackish or salt from the tide, and which for a 200 million gallon plant will make the cost of transportation alone about $18.67 per million gallons for merely carrying the water from the pumps to the city limits.

With a large withdrawal of water from the Hudson at Poughkeepsie for New York's supply the limit of brackish water would be moved farther upstream, particularly in a cycle of years of low rainfall. It is stated that in the low rainfall cycle culminating in 1883, the water became brackish at times as far up as Kingston. This could probably be remedied by regulating the flow of water from the large Adirondack storage reservoirs which have been often proposed.

The project of filtered water from the Hudson possesses the advantage of a volume of water in the river sufficient, with the summer outflow that could be obtained from the future Adirondacks storage reservoirs, for almost indefinite extension as New York grows, and is in line with the proposed new supplies for Philadelphia, Pittsburgh and Cincinnati. Each of these cities has turned to the river near by, and, like certain of the largest European cities, sought to purify the water to be obtained therefrom under the developments of modern science. In Philadelphia and Pittsburgh, the projects for mountain water from distant sources were abandoned in favor of filtration of river water.

The art of filtration, stimulated by the wonderful developments of recent bacteriological research, has made great advances within the past ten or fifteen years, so that with care to locate the intake away from gas works or factory wastes or the vicinity of an outfall sewer, no question or doubt remains about the good taste, clearness, purity and healthfulness of properly filtered water from a river like the Hudson. *The only doubt is that arising from the possibility of incompetent supervision.* This, for the case of Boston and in the investigations of the Massachusetts State Board of Health when the possi-

bility of filtered water from the Merrimack was considered as a source for the new Metropolitan supply, was thought worthy of serious consideration, particularly when, on the other hand, good unpolluted water was obtainable from a flowing stream.

For twenty-seven years Poughkeepsie has taken its water supply from the Hudson, and while the methods and appliances in use there are less perfect than those since devised, the source of supply has in general proved satisfactory, although the water has occasionally tasted a little salt, according to entirely credible testimony. The original filtration works at Poughkeepsie were devised by Mr. James P. Kirkwood, who designed the original Brooklyn water works, after a careful study of the methods of filtration at that time practised in Europe. A new and improved filter bed was constructed for the extension of the Poughkeepsie works about 1895.

The City of Hudson, N. Y., also has for about twenty-five years taken its water from the Hudson river and filtered it.

The City of Albany, during the past three years, has gone to the Hudson river for an increase to its water supply, and from the designs of Mr. Allen Hazen, has built what is, so far, the most extensive filtration plant in use in the United States, at an expense of half a million dollars. This is capable of filtering 15 million gallons of water per day. The works have thus far been in use about seven months and give every indication of proving eminently successful.

London, Hamburg, Altoona, Rotterdam, Breslau and Berlin are noteworthy examples of great cities abroad which secure excellent supplies from rivers, and in several instances these foreign rivers are less promising as sources of drinkable water than the Hudson, with its large volume of flow.

The long continued success of the Poughkeepsie works gives proof that fresh water, seldom salt or brackish from the sea tide, and when so, not to a occasional slight turbidity, and needing only proper filtration to purify it from the pollution of the cities and of the shipping. Judging by this experience of Poughkeepsie and being well assured that, at a practicable expense, enough fresh water could be discharged from future reservoirs in the Adirondacks to supply more than the amount diverted for New York and at the same time to remove the menace of brackish water from the proposed intake above Poughkeepsie, *we may feel entire confidence in the adequacy of a supply of filtered water taken from the Hudson above Poughkeepsie to supply all of New York's needs.*

Location of Intake and Pumps.

The flood tide sets upstream with a strong current at Poughkeepsie, and the city's sewage may, therefore, be carried northward for a brief period each day. This question, we are informed, was carefully investigated by the distinguished chemist, Dr. Thomas M. Drown, now President of Lehigh University, who found that the large volume of river flow quickly diluted the Poughkeepsie sewage almost beyond the power of analysis to discover or distinguish beyond the other similar material diffused through the river.

Possibly extended research would show that after the construction of the proposed Adirondack reservoirs this Hudson river water could be taken from a point further down the river than Poughkeepsie, thereby saving expense of conduit. Some three miles below Poughkeepsie is an inviting site for investigation, where possibly 500 acres of terrace about a hundred feet above the river and close beside it would afford an economical site for the filters and the sedimentation basins, although the elevation is such that the water would require twice pumping. But, to be on the safe side in this preliminary study, the point of intake and location of pumping station will be assumed above the City of Poughkeepsie at, say, about one-half mile, or possibly one mile upstream from the Poughkeepsie Bridge. An addition for water storage in the Adirondacks will be added to the estimate.

Location of Filters.

A wide range of sites gives ample opportunity for choice. The most natural location would be close beside the river, as at Albany, near to the proposed pumping station already mentioned, but the lay of the land there is not favorable, particularly for a comprehensive project with large future extensions. While it is possible that by some allowable incroachment upon the river, by use of deeper sedimentation basins and by taking a long water front, sufficient land for a 100 million gallon filter plant might be secured, it would be at large expense; the shore immediately above Poughkeepsie is so bold and rocky that the outlook is almost hopeless for obtaining room on the river bank for anything more than an intake and a large pumping station. It has appeared prudent to let the rough preliminary estimate rest on a plan which should give a wider selection of sites.

Economy will after all perhaps be best served by placing the filters on high ground well back from the river in a region where abundant land for future extension can be cheaply had, and locating the pumps all on the river margin, where coal can be unloaded from barges into pockets, whence it

can slide by gravity or be moved by conveyors to mechanical stokers at a minimum of labor cost. On the river margin too, ample water for engine condensation will be cheaply secured.

The filters, if back from the river will, for economy of pumping, best be placed upon land so elevated that after the water is once lifted from the river, no further pumpage will be required, because the necessary losses by fluid friction through the valves and passageways of the pump, and certain of the losses of mechanical friction in the pump are independent of the lift and are doubled with twice pumping. The topographical map gives hope of finding suitable land at proper elevation for so locating the filters. If other considerations prove of greater weight than an elevation suitable for a single pumping, then there are various broad level areas available by a second pumping into the high-level aqueduct, or available without the second pumping for a lower terminal pressure. With once pumping the cost will be a little smaller than with twice pumping, and as there is a fair possibility of ground at suitable elevation, we will base the figures on pumping but once.

High Pressure Supply.—To make the result strictly comparable with the proposition of the Ramapo Company to deliver water from the Esopus at a hydraulic grade of 300 feet above tide at the city limits, and also to meet the claim of certain prominent underwriters, that water at this high head would be of great advantage to the safety of New York, and to further put the filtration project on a fair basis of comparison with the project of water supply from the Ten Mile and Housatonic rivers delivered under a pressure of 300 feet head at the city limits, I have first estimated upon outline plans for a high-level supply of filtered water. Evidently, the water can be pumped to only two-thirds this height at Poughkeepsie, and delivered at the present level of Central Park or Jerome Park reservoirs at much less expense.

Location of Filter Beds.—As a basis for estimates, without in any way implying that this spot is the best that can be found, I have marked on the map a site where the ground is at favorable elevation, near the line of the railroad, about 1½ miles northerly from Fishkill Village and 4 miles east of the Hudson, and about 12 miles along the conduit southerly from Poughkeepsie. This particular site is a rounded hill-top of about 300 acres of grass land and tillage, with indication that there is a considerable depth of impervious earth. A well on one side of the area is said to have been sunk about 20 feet deep without reaching ledge, and on the other side of the area is a well 30 feet deep without rock having been reached. This site would

require a good deal of expensive grading in leveling off and preparing the beds. Whether this or some other site on the same general line is chosen, need not affect the cost materially. To pick out the best site would require accurate levels, contour maps, borings, test-pits and examinations, which under my limitations of time and expense are impossible.

The works at the filter plant upon which I have estimated, briefly outlined, would consist of:

1st. The delivery pipes from the pumps would discharge into a series of subsidence basins of capacity sufficient for 24 hours storage in transit, arranged for continuous delivery, and with additional basins of 25 per cent. surplus capacity, which can be idle or in use. The whole arranged to serve as an equalizer of the pump delivery to the extent of 5 or 10 feet rise or fall.

2d. Covered filter beds, similar in all respects to those constructed for Hudson river water at Albany. Each filter to have 0.7 acre net filtering area, filtering 3 million gallons per acre per day at nominal rate, or 2.1 million gallons per filter.

3d. A small, clear-water gallery, holding about an hour's total delivery, and serving mainly as a small equalizing chamber at the head of the aqueduct or conduit line.

4th. In view of the ultimate extent of the works, a railroad siding may profitably be run to them from Brinckerhoff.

Cost of Pumping Plant.

The cost of the works and the expense of operation is estimated approximately as follows, the intention being that, in absence of definite locations and accurate surveys, prices should be large enough to cover contingencies, and yet as small as appears safe, so that this filtration and pumping project may not suffer in comparison with the gravity projects.

The estimate will be made on the basis of an average use of 200 million gallons of water per day; but as was done with conduit lines from the Ten Mile, etc., an extra capacity of 25 per cent. will be added as a reserve to allow for a part of the pumping plant, and for part of the filters being continually in idleness for overhauling or repairs, and also to provide for the rate of draft in some months and some weeks largely exceeding the average for the year.

High Level Filtration Project, Lift of Pumps—

Hydraulic grade delivery at northern city limits of New York assumed, etc.....		300 feet above sea.
Assumed average hydraulic gradient for entire conduit line, 2.0 feet per mile; distance pumps to city limits, 61.5 miles.....		123 feet.
Assumed drop at filters from pump delivery for aeration.....	5 ft.	
From high-water level in subsidence basin to lowest level of same.....	8 ft.	
Assumed loss in passages getting water on to filter.	1 ft.	
Loss of head in filtering.....	4 ft.	
Assumed loss of head taking water away.....	1 ft.	
Assumed fluctuation in clear water-gallery	5 ft.	
	24 ft.	
This being a preliminary estimate, and exact location uncertain, more drop is provided for than commonly found necessary.....		24 feet.
Total dynamic head on pumps....		447 feet.
Lift used in estimates.....		450 feet.
Average daily pumpage.....		200 million gallons.
Pumps to be provided, including reserves of total capacity for daily pumpage of..		250 million gallons.

Probable cost of land for pumping station and coal pocket on river margin	$50,000
Wharf, river wall, intake crib and two 8-foot submerged conduits to pump wells, cost influenced largely by location, say.	150,000
Two pumping stations—separated 100 feet, fireproof construction, each complete with its own boilers and accessories for engines of 125 millions capacity, including foundations for engines. Cost largely dependent on substrata as found, say	500,000

Pumping engines—In units of 15 to 20 millions, triple expansion of high duty type, giving when new, at duty trial, 150 million foot-pounds per 100 pounds coal, and of such quality of material, workmanship and design as to yield when ten years old, in daily use with moderate care, 110 million foot-pounds per 100 pounds coal.

Prices for heavy pumping machinery at present are somewhat high and unsettled. A study of certain recent contract prices and rough estimates, kindly prepared for me in the offices of two of the largest builders of pumping machinery in the United States, indicate that for 450 feet lift and in units of from 15 millions to 20 millions, a cost of $14,000 per million gallons of capacity is a safe figure to-day for pumping engines of the highest grade, including piping and all accessories, including also steam boilers of either the water tube or Manning type adapted for 150 to 160 pounds steam, and including settings; all erected and complete—

250 million gallons capacity, @ $14,000 for 450 feet lift	3,500,000

Horse-power of engines—To lift 200 million gallons per 24 hours, 450 feet requires 15,770 work horse-power. Engines giving 92 per cent. mechanical efficiency at duty trial could be relied upon, with moderate neglect and with some plunger slip and valve leakage, after ten years' use, to give at least 75 per cent. mechanical efficiency, or 85 per cent. with good care.

At 75 per cent. efficiency, indicated horse-power required, ten years old	21,000 horse-power.
At 90 per cent. efficiency, indicated horse-power required, new	17,500 horse-power.

Total	$4,200,000
Contingencies and supervision, at 15 per cent	630,000
Estimated total cost of pumping plant, exclusive of delivery main, for land, structures and appliances	$4,830,000

There is great variation in different cities having large pumping plants in the fuel and labor costs, when reduced to a uniform basis, depending in part upon quality of management, whether or not the station is overmanned and the boiler-room economies closely watched, and in part upon how

much of the plunger displacement is lost by slip. By an analysis of the actual pumping station expenses in several large cities, the following appears a fair allowance with plant ten years old, viz.:

$0.035 per million gallons raised 1 foot high,

or for 450 feet high × 0.035 =.....$15.75 per million gallons.

(With best management the cost need be but about ⅔ this sum.)

This economy could be easily reached or exceeded in practice at the proposed site with low-cost coal, cheaply handled, and pumps in large units with high lift, run at full nominal capacity.

Cost of Albany Filter Plant.

In advance of working out the details of a definite design, no better basis for this estimate can be found than the cost of the plant recently constructed at Albany, under the supervision of Mr. Allen Hazen, for filtering Hudson river water; adding whatever may appear proper for less favorable conditions at the site chosen for the New York project. These Albany filters, consisting in all of eight beds, of which one is supposed idle for cleaning, are called of 15 million gallons daily capacity, but including reserve have a greater capacity. Each of these eight beds has 0.7 acre of net surface, and at the nominal rate of 3 million gallons per acre per day, the eight beds have 16.8 million gallons total capacity.

Cost of Albany Filters.

	Total cost for Albany Plant.	Cost per mil. gals. of total capacity daily.
The actual total cost of covered filters, 16.8 million gallons' capacity, exclusive of pumps and land, was about	$255,000 00	$15,200 00
Cost of sedimentation basin, about 24 hours, or 14.6 million gallons' capacity, about	60,000 00	3,600 00
Pure water reservoir, capacity 600,000 gallons, about one hour's capacity, about	9,000 00	540 00
Total, including extra work, laboratory building, sand, gravel, etc., exclusive of pumping plant and conduits	324,000 00	19,340 00
Supervision, about	20,000 00	1,190 00
Land	8,200 00	490 00
Total, including land	$352,200 00	$21,020 00

The total cost of the Albany plant, exclusive of land, pumps and conduits was thus about $20,500 per million gallons of total capacity.

ROUGH ESTIMATE OF COST OF SLOW SAND FILTRATION PLANT.

Located on High Ground Between Poughkeepsie and Fishkill.

Total capacity	250 million gallons per day.
Working capacity	200 " " " "
On same basis as at Albany, cost for 250 million gallons total capacity, 250 × $20,500	$5,125,000
If to this, in addition to ordinary contingencies, we add 15 per cent. for less favorable ground requiring more excavation, higher cost of labor and building material, and less favorable freight rates	769,000
Total	$5,894,000
Add for railroad siding, say	30,000
Add for shops and storage buildings; for small electric light and power plant; for small steam plant and pumps for wash-water; for two residences for superintendent and bacteriologist; for fences, roads and concrete walks, say	150,000
	$6,074,000
Add to this the customary 10 per cent. for contingencies of a preliminary estimate, supervision being included above in the Albany data	607,000
	$6,681,000
Add for land 175 acres at $200, including legal expenses	35,000
Total cost of plant on Albany basis	6,716,000

Checking the above roughly by the estimated cost of the Torresdale plant proposed for Philadelphia, which was designed for 300 million gallons working capacity, we have—

Estimated Cost of Philadelphia Torresdale Plant.

Excavation	$314,556
Piping	504,355
Raw-water conduit	224,964
Filtered water conduit	77,976
Water-mains, sand-washers	13,000
Drains	87,874
Sand-washers	21,600
Filter beds	5,159,087
Sedimentation reservoir	1,712,420

Tramway and equipment		$54,800
Residence, laboratory, etc.		167,300
Fencing, side tracks and clearing up		53,000
Divide total cost by total daily capacity	300)	$8,390,932
Cost per million gallon capacity, of Torresdale plant		$27,970
Proposed capacity, New York project		200
		$5,594,000
Add 15 per cent. contingencies and supervision		839,000
		$6,433,000
Add for land		35,000
Total cost for plant of 200 million gallons daily capacity on Philadelphia, Torresdale, basis		$6,468,000

For a preliminary figure to aid in deciding whether the filtration project for New York is of sufficient promise to warrant further study, we thus have—

On Albany basis	$6,716,000
On Philadelphia basis	6,468,000
And we appear justified in calling probable cost of filters and accessories, about	6,800,000

Cost of Operation of Filters.

The cost at Poughkeepsie has for twenty years averaged ..... $3 per million gallons,
—including cost of ice removal and more frequent scrapings, because of algæ, than would be necessary with the covered filters proposed in this estimate.

Average cost in London, England, for seven years (per Hazen) with wages of laborers averaging $1 per day was found to be ..... $1.24 per million gallons.

The Philadelphia Commission of Engineers, Messrs. Hering, Wilson and Gray, in October, 1899, adopted as a safe figure for the proposed Torresdale plan on Delaware river, for slow sand filtration ..... $3 per million gallons.

The Albany plant has not yet been long enough in operation to give full data on cost of operation.

I have adopted as a safe outside figure for the cost of operation of this large plant proposed for New York City, with modern appliances for filtering Hudson river water, the same figure reported for actual average cost for 20 years at Poughkeepsie, namely..... $3 per million gallons. This figure, moreover, is the same as that reported for the proposed Torresdale filters on the Delaware, by the Philadephia Board of Experts.

There is a fair chance that $2.50 per million gallons would be enough to cover the actual cost of operation.

Conduit Line from Poughkeepsie to New York.

The conduit, while intended to carry an average of 200 million gallons daily, is given a size sufficient, when clean, to carry 250 million gallons. This surplus is intended to provide for occasional deliveries at above the average rate in those months like February or September, when the consumption is found greatest, and to also provide for impairment of the carrying capacity of the masonry aqueducts from lack of cleaning, similar to the 15 per cent. impairment now suffered by the New Croton Aqueduct.* For steel pipes the coefficient of flow is taken the same as that actually found for the East Jersey conduit above Pompton. A hydraulic gradient of 2.0 feet per mile is assumed throughout. Possibly the final study will show some different value to be a little more economical when interest on conduit is carefully balanced against the total cost of pumping.

The location, as outlined below, is to be considered only as a first approximation and as a safe basis for a rough estimate. Naturally the closer a location is studied the cheaper the line that can be found. The problem of the most economical and permanent location for conduit from a filtration plant was kept in view during a rapid reconnaissance for conduit lines from the Esopus and for gravity supplies of surface water from the regions north and northwest from the Croton Watershed. Although an "all pipe" line could be run, the "cut and cover" masonry aqueduct, when ground of the right elevation can be found, is very much the cheapest and most satisfactory form of aqueduct, and an effort was made to secure a location giving as much of this type of construction as possible. The country east of Fishkill and Peekskill is so high, so extremely mountainous and with such bold precipitous slopes along the river that only a little study is required to show that for about ten miles the conduit line can best proceed in tunnel.

* With filtered water, the impairment of carrying capacity due to slime would probably be very small, and a slight unfairness to the filtration project may follow from estimating on the same size of aqueduct as in the Ten Mile and Adirondack projects, but on the other hand the large cost of covered reservoirs would tend toward larger conduits and less storage.

The high level line for delivering water at elevation 300 at city limits can apparently be more cheaply and more quickly constructed than the low level line.

Starting from the pumping station at Poughkeepsie with this trial line and quickly seeking high ground to lessen the pressure on the pipe, three steel pipes of 7 feet 3 inches diameter are run for about two miles until high ground is reached, and thence for about ten miles two pipes of 8 feet 5 inches diameter run to the filter plant; delivering with cross-connections into the four main divisions of the sedimentation reservoirs.

From the pure water gallery of the filter plant the two steel pipes continue south past Brinckerhoff for about two miles until ground is reached at suitable elevation for a cut and cover masonry aqueduct; this follows the contour of the hillside four miles to the entrance of a tunnel, which is in all about ten miles long. There is near the middle of tunnel one deep valley and there are other high valleys crossing the line, which afford convenient sites for shafts.

The tunnel emerges near the head of what, in certain of the other trial lines, is called the proposed Holman's Mills Reservoir, and thence by a cut and cover aqueduct, the conduit follows the contour of the hillside along to a point near the Old Croton Dam, crosses the Croton Valley by a siphon and proceeds down the Pocantico Valley, past Merritt's Corners, and along the contours of the east side of the Saw Mill river valley, finding much of the way suitable location for cut and cover aqueduct which can be built at about half the cost of steel pipe lines.

The conduit line follows the top of the ridge as it nears the city limits, until about half a mile north of the city limits, it delivers into the proposed Park Hill high pressure distributing reservoir at about elevation 303.

A large high level distributing reservoir like that proposed at Park Hill is essential to the proper operation of any high pressure gravity supply, and a rough estimate of the cost of the Park Hill Reservoir amounting to $3,696,000 is included, although nothing of the kind was provided for in the plans of the high pressure supply tendered by the Ramapo Company.

An approximate estimate of the cost of this 250-million-gallon aqueduct is given on the next page, and for convenience of comparison with the Ten Mile and Housatonic scheme, other estimates are carried out for the same capacities as provided for in the studies for aqueducts for utilizing the entire capacity of the Ten Mile and the Upper Housatonic rivers.

TABLE No. 69—FILTRATION PROJECT.

(From Rough Brief Reconnaissance Only.)

Approximate Cost of Aqueduct Lines. "High Pressure" Line from Poughkeepsie to City Limits.
Distance along Conduit Lines, 61.5 Miles.

Assumption—Hydraulic Gradient 2.0 Feet per Mile. Distance, pumps to filters..... 11.8 miles. Filters to City limits..... 49.7 " Total length aqueduct..... 61.5 miles. Hyd. grade, City limits..... 300 ft. " outlet of filters..... 400 " " at pumps, 444, or, say..... 450 "	Capacity (clean), 250 Million Gallons per 24 Hours. For Average Use of 200 Million Gallons.		Capacity (clean), 500 Million Gallons per 24 Hours. For Average Use of 400 Million Gallons.		Capacity (clean), 1,000 Million Gallons per 24 Hours. For Average Use of 800 Million Gallons.	
	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.
	Cost per Unit.	Total Cost.	These Two Columns are Added for Comparison with Ten-Mile and Housatonic and to Show Comparative Economy by Making Masonry Portion in One Line Instead of Two.			
Full size from the start—						
26 miles C. and C. masonry ag.....	$137,000	$3,562,000	$177,000	$4,602,000	$248,000	$6,448,000
6 miles arch lined tunnel.....	307,000	1,842,000	403,000	2,418,000	563,000	3,378,000
4.5 miles side lined tunnel.....	248,000	1,116,000	329,000	1,481,000	456,000	2,052,000
Steel pipes put in at beginning, 1 line only, except for 7 ft. 3 in. pipe, of which 2—						
12 miles steel pipe, $\frac{7}{16}$-in. thick (2, 4 and 7 lines).	8 ft. 5 in. diam. 160,000	1,920,000	8 ft. 5 in. diam. 160,000	1,920,000	8 ft. 11 in. diam. 176,000	2,112,000
10 miles steel pipe, $\frac{8}{16}$-in. thick (2, 4 and 7 lines).	8 ft. 5 in. diam. 177,000	1,770,000	8 ft. 5 in. diam. 177,000	1,770,000	8 ft. 11 in. diam. 188,000	1,880,000
3 miles steel pipe, $\frac{3}{4}$-in. thick (3, 5 and 10 lines)	7 ft. 3 in. diam. 212,000	1,272,000	7 ft. 9 in. diam. 232,000	1,392,000	7 ft. 9 in. diam. 232,000	1,392,000
Present value of additional pipes, put in as growth requires—						
2d line of steel pipe, 22 miles.....	In 6½ yrs.	3,045,000	6½ yrs.	3,045,000	7½ yrs.	3,198,000
3d line of steel pipe, 22 miles.....	.....	.....	13 "	2,513,000	15 "	2,562,000
4th line of steel pipe, 22 miles.....	.....	.....	20 "	2,043,000	23 "	2,023,000
5th line of steel pipe, 22 miles.....	.....	.....	.....	.....	30½ "	1,621,000

6th line of steel pipe, 22 miles	.....	.....	.....	.....	38 "	1,298,000
7th line of steel pipe, 22 miles	.....	.....	.....	.....	46 "	1,025,000
2d for extra heavy pipe near pump, 3 miles	3d in 9 yrs.	487,000	10½ yrs.	1,021,000	10½ "	1,021,000
3d for extra heavy pipe near pump, 3 miles	.....	.....	21 "	374,000	21 "	748,000
4th for extra heavy pipe near pump, 3 miles	.....	.....	.....	.....	32 "	540,000
5th for extra heavy pipe near pump, 3 miles	.....	.....	.....	.....	43 "	391,000
1,600 linear feet shaft excavation	100	160,000	125	200,000	150	240,000
16 siphon chambers	10,500	168,000	14,000	224,000	21,000	336,000
Bridges, culverts, blowoffs, manholes, etc., 53 miles	15,000	795,000	21,000	1,113,000	30,000	1,590,000
Right of way C. and C. 28 miles, 100 ft. wide	3,000	84,000	3,000	84,000	3,000	84,000
Right of way tunnel, 50 feet wide, shafts	.....	24,000	.....	25,000	.....	26,000
Right of way pipe lines, 25 miles	60 ft. wide 1,800	45,000	90 ft. wide 2,700	68,000	120 ft. wide 5,600	90,000
	.....	$16,290,000	.....	$24,293,000	.....	$34,055,000
Add 15 per cent. for contingencies and supervision	.....	2,443,000	.....	3,644,000	.....	5,108 000
Total	.....	$18,733,000	.....	$27,937,000	.....	$39,163,000
Park Hill Reservoir, North Basin	.....	3,696,000	.....	3,696,000	.....	3,696,000

Cost of an Average Quantity of 200 Million Gallons of Filtered Water Per Day from Pumping Station at Poughkeepsie to New York City Limits.

(From Brief Reconnaissance Only.)

The table containing a summary of this computation has been transferred back to Table 8, page 105, of the main report, for convenience of comparison with the other possible sources. By reference to this it will be seen that the total estimated cost per million gallons for filtered water from the Hudson, delivered at the city limits at an elevation of 300 feet above sea level, including all depreciation, interest, operating expenses and an annual contribution to a sinking fund sufficient to pay off the bonds for the entire cost in 40 years, is $47.10 per million gallons.

With the best of engine-room and boiler-room economy, the cost of pumping could be reduced materially, and it is not impossible that a close estimate based on accurate surveys would as a minimum bring the cost down to $40.00 per million gallons. Under most unfavorable conditions it could hardly be so much as $50 per million gallons including the sinking fund.

Omitting the sinking fund requirement the cost becomes $40.77 per million gallons and omitting the Park Hill Reservoir, for which no equivalent is found in the Ramapo proposition, the cost becomes only about $37.50 per million gallons.

Comparison of Filtration Project with the Ramapo Company's Proposition.

First, the cost of the filtered water, when put on the same basis, would be only about $37.50 per million gallons as against the Ramapo Company's water at $70 per million gallons.

For comparison with the Ramapo Company's proposition, it is to be noted that the estimated cost of $40 to $50 includes the contribution of a sinking fund which shall pay off the bonds and recover the entire cost of the works in 40 years.

That it gives near the city limits a high pressure distributing reservoir nearly as large as the largest reservoir in Central Park, as a safeguard against brief interruptions.

That there is certainty that the supply of 200 million gallons could be furnished at all times.

That although there might be some prejudice against water from the Hudson, this water would, as a matter of fact, probably be more pure and more clear than the unfiltered water from Esopus Creek, which has some villages that are quite populous in summer close to the banks of the stream. The Esopus is for brief periods much discolored by the wash of red clay or similar fine material in the spring flood season and for a day or two after heavy rains, and this suspended material does not subside quickly when the water is in quiet storage. The Esopus water is, however, commonly an unusually clear, soft, pure surface water.

The works of the Poughkeepsie filtration project could be constructed just about as quickly as the proposed dams, reservoirs and conduits from Esopus Creek.

Possibility of Salt Water at Poughkeepsie.

The minimum volume flowing in the Hudson river at Poughkeepsie in time of extreme drought is a question of vital interest in this connection. From the published statements made in years past by George W. Rafter, C. E., who has made special studies of the flow of the Hudson river, and of the remarkable possibilities for cheap storage upon its head waters (see Bulletin No. 24, U. S. Geological Survey, Water Resources of the State of New York, Part 1, page 82; also Transactions American Soc. C. E., Feb., 1900, p. 189), we learn:

The smallest mean monthly flow gauged at Mechanicsville from 1888 to 1899, inclusive, was in August, 1899, and was at the rate of 0.31 cubic feet per second per square mile. July and August, 1888, gave monthly means of 0.34 and 0.38 respectively. From page 97, of Bulletin 24, it appears that the smallest run-off for a week, after correcting for diversion at the State dam, was 0.29 cubic feet per second per square mile. All of these figures probably include storage water sent down from the Lumbermen's dams in the Adirondacks to benefit the water-power below. and so cannot safely be applied to the watershed entering below Mechanicsville.

The " catchment area " or drainage area at Mechanicsville, is per Mr. Rafter, about 4,500 square miles.

The drainage area of the Hudson at Albany is about 8,240, and at Poughkeepsie, is not far from 11,800 square miles.

We shall probably not err very much if we follow Mr. Allen Hazen in his estimates of the flow at Albany, and call the minimum flow from the Mohawk and other tributaries below Mechanicsville, 0.15 cubic feet per second per square mile, and 0.24 down to Glens Falls or Mechanicsville.

This gives: 4,500 square miles × 0.24 = 1,080 cubic feet per second.
7,300 square miles × 0.15 = 1,100 " " " "

2,180 cubic feet per second =

1,410 million gallons per 24 hours at Poughkeepsie in extreme drought. This plainly gives a small margin of safety against saltness consequent to the neglecting the tributaries below the Mohawk, the ebb and flow of tide at

Poughkeepsie if any large volume is to be withdrawn. However, we have reliable evidence that in seasons of extreme low river flow in the past saltness has been noticed at Poughkeepsie and above. The problem of just how much water must be added to keep the river always fresh is complicated and requires much time for computation, so to be safe we will add bodily to our estimate the entire sum of $2,606,559, stated by Mr. Rafter at bottom of page 176, or again on page 194, item 14, in the report of the New York State Engineer for 1895, as the cost of the storage reservoirs required to give a minimum flow of 4,500 cubic feet per second at Mechanicsville. This is an unsatisfactory form in which to leave this question of Adirondack storage in its bearings on filtration at Poughkeepsie, for obviously the great benefits to navigation and to the industry of the State should carry a part of the cost of these reservoirs and the capitalized cost of their maintenance. We will enter up $2,500,000, as a charge against the filtration project for 800 million gallons per day and $1,000,000 for 200 million gallons per day, and add a small sum for care of these reservoirs in the maintenance charges, and so display the figures that they can be readily modified after further study.

And whenever this great storage system, so often urged and so well described by Mr. Rafter in the Annual Report of the New York State Engineer to the General Assembly for 1895, is built, the least flow, according to Mr. Rafter will be more than five times the largest draft that I have anywhere considered, namely, more than five times 750 million gallons per day.

LOW PRESSURE FILTRATION PROJECT.

For the purpose of determining what relative economy may exist in a plan which, instead of delivering the water at city limits at an elevation of 300 feet, delivers it at the level of the Jerome Park Reservoir (elevation 131.5 feet above sea), the following estimates were made.

These estimates, except as stated below, have the same basis used in the foregoing analysis of the cost of the "high level":

Hydraulic grade at northern city limits of New York	135	feet above sea.
Assumed average hydraulic gradient for entire conduit line, 2.0 feet per mile; distance, pumps to city limits, 58.7 miles	117.4	feet.
Assumed drop in passing through filters	24.0	"
Total working head on pumps	276.4	feet.
Lift used in estimates	280.0	"

This lift is 170 feet less than that required for the "high level" filtration project.

Cost of Pumping.

Pumping Plant (similar to that for high level):

Land for pumping station, etc. (same as "high level")	$50,000
Wharf, crib and intake works (same as "high level")	150,000
Two pumping stations, etc. (same as "high level")	500,000
Pumping engines—in units of 15 to 20 millions, triple expansion, high duty type—250 million gallons capacity, at $10,500 per million gallons, for 280 feet lift	2,625,000

In terms of horse-power—

To lift 200 million gallons daily 280 feet requires 9,830 work horse power.

At 75 per cent. efficiency, indicated horse-power required 10 years old, 13,100 H. P.
At 90 per cent. efficiency, indicated horse-power required when new, 10,900 H. P.

Estimated total cost of pumping plant, exclusive of delivery main	$3,325,000
Contingencies and supervision, at 15 per cent	499,000
Total	$3,824,000

Cost of Filtering.

The cost of slow-sand filtration is independent of elevation, except because of the required elevation of the filters being much lower on the low level than on the "high level" project, they will be built on a different tract of ground, and this may modify the cost of construction according as the local conditions prove more or less favorable. It is assumed that the cost for filters will be the same in either location, or a total of $6,800,000.

Conduit Line.

As in the case of the "high level" project, the route outlined below, as a possible location for this conduit, is to be regarded merely as a first approximation, which future surveys may show must be shifted more or less to adjust it to local conditions, and yet it is probably near enough to the best possible route to serve all needed purposes of this estimate.

Starting from the pumping station at Poughkeepsie, the location is assumed to be the same as for the first three miles of the "high level" line; first reaching out for the high ground to the eastward and then swinging around the thickly settled portion of Poughkeepsie, it takes a course almost due south to the filtration works provisionally located on a rise of ground one mile northerly from Fishkill Village. The map contours indicate that the desired elevation of from 240 to 260 feet can there be obtained, and an inspection of the surface indications at that site gave prospect of considerable depth of earth over the ledges and appeared favorable for cheap work in grading this tract. The force main to this point would consist of two lines of riveted steel pipes 8 feet 5 inches diameter, and with thickness of shell varying from 3/8 inch to 11/16 inch, to meet the different conditions of pressure.

If surveys and borings should prove the ground just suggested to be unsuitable for the filters, and if selection be made of lower ground, such as the tract near Camelot heretofore mentioned, then with the pumps located up river from Poughkeepsie, it might prove cheaper and better to pass under that city in tunnel and to construct a section of masonry aqueduct for some distance beyond towards the filters. No definite conclusions can be reached in this matter without precise instrumental surveys.

From the first-named location for the filters it appears possible to construct about 4½ miles of "cut and cover" masonry aqueduct, which shall receive the pure water by gravity directly from the collecting galleries of the filters. Thence an air-line tunnel, about 16 miles long, under the high and rugged country to the eastward of Peekskill and Fishkill has been assumed for the present as the line on which to base estimates. Tolerably good shaft sites can be found on this air-line location at intervals of about one mile, though five of the shafts will range from 400 to 700 feet in depth and it is not unlikely that precise surveys would show some advantage to be gained by swerving from a direct line at those places.

The southern portal of this tunnel will be at the head of the New Croton Reservoir. The hydraulic grade elevation will there be about 190, or ten feet lower than the proposed flow line of the new reservoir. For the next three miles, the conduit will be within the limits of Croton Lake, crossing the valley of Croton river, and must consist of steel pipe. Between the south shore of Croton Reservoir to city limits it is proposed to have the aqueduct wholly in tunnel, and to that end the route should be sought which offers the firmest rock formation and the fewest complica-

tions as to right-of-way, without too much deviation from a direct line. This might be best found on a location near to and practically parallel to the New Croton Aqueduct, or it is quite probable that, under present conditions, more favorable ground would be found generally from one mile to one and one-half miles eastward from the Croton tunnel.

Extending this tunnel 1.7 miles south from city limits, brings it to the Jerome Park Reservoir, into which delivery would be made for purposes of distribution.

An approximate estimate for this aqueduct is given on the next page. This provides for an average daily delivery of 200 million gallons, plus a margin of 25 per cent.; and, in addition, estimates are carried out for the larger average deliveries of 400 and 800 mililon gallons per day, for convenience in making comparison with the Ten Mile and Housatonic river projects.

TABLE NO. 70.

(From Rough Brief Reconnaissance only.)

Estimated Cost of Low Pressure Conduits, Poughkeepsie to New York City Limits.

Distance from Pumping Station, ½ Mile Above Poughkeepsie Bridge, to City Limits = 58.7 Miles.

Assumptions— Pumps to filters, force main, 12.3. Elev. at Jerome Park Reservoir, 131.5 ft. Elev. at city limits, 1.7 miles north, 135 ft. Hyd. grade at filters, 228.0 ft. Total lift at pumps, 280 ft.	Capacity (clean), 250 Million Gallons per 24 Hours for Average Use of 200 Millions.		Capacity (clean), 500 Million Gallons per 24 Hours for Average Use of 400 Millions.		Capacity (clean), 1,000 Million Gallons per 24 Hours for Average Use of 800 Millions.	
	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.
	These two columns are added for comparison with Housatonic and Ten Mile and to show comparative economy by making masonry portion in one line instead of two.					
Full size from the start—						
4.3 miles C. and C. aqueduct	$137,000	$589,100	$177,000	$761,100	$248,000	$1,066,400
20 miles arch lined tunnel	307,000	6,140,000	403,000	8,060,000	563,000	11,260,000
19.1 miles side lined tunnel	248,000	4,736,800	379,000	6,283,900	456,000	8,709,000
Steel pipes put in, 1 line at a time—						
12.3 m. steel pipe $\frac{7}{16}$ in. thick (2, 4 and 7 lines)	8 ft. 5 in. diam. 160,000	1,968,000	8 ft. 5 in. diam. 160,000	1,968,000	8 ft. 11 in. diam. 176,000	2,164,800
2 m. steel pipe $\frac{8}{16}$ in. thick (2, 4 and 7 lines)	8 ft. 5 in. diam. 177,000	354,000	8 ft. 5 in. diam. 177,000	354,000	8 ft. 11 in. diam. 188,000	376,000
1 m. steel pipe $\frac{11}{16}$ in. thick (2, 4 and 7 lines)	8 ft. 5 in. diam. 198,000	198,000	8 ft. 5 in. diam. 198,000	198,000	8 ft. 11 in. diam. 210,000	210,000
Present value of additional pipes discounted at 3 per cent., put in as growth requires—						
2d line of steel pipe 15.3 miles, deferred	6½ yrs.	2,080,500	6½ yrs.	2,080,500	7½ yrs.	2,204,000
3d line of steel pipe 15.3 miles, deferred	.....	.....	13 "	1,716,000	15 "	1,725,000
4th line of steel pipe 15.3 miles, deferred	.....	.....	20 "	1,395,000	23 "	1,387,000
5th line of steel pipe 15.3 miles, deferred	.....	.....	.....	.....	30½ "	1,117,000

6th line of steel pipe 15.3 miles, deferred	.....	.....	.....	....	38 "	894,700
7th line of steel pipe 15.3 miles, deferred	.....	.....	.....	.....	46 "	706,300
6,700 linear ft. shaft excavation	100	670,000	125	837,500	150	1,005,000
5 siphon chambers	10,500	52,500	14,000	70,000	21,000	105,000
Bridges, culverts, blow-offs, manholes, etc., 20 miles	15,000	300,000	21,000	420,000	30,000	600,000
Right of way, C. and C., 4.3 m. 100 ft. wide	3,000	12,900	3,000	12,900	3,000	12,900
Right of way, tunnel, 50 ft. wide, shafts extra	.....	90,600	.....	93,300	.....	98,800
Right of way, pipe line, 15.3 miles	60 ft. wide 1,800	27,500	90 ft. wide 2,700	41,300	120 ft. wide 3,600	55,000
		$17,219,900		$24,291,500		$33,696,900
Add 15 per cent. for contingencies and supervision, etc.	.....	2,583,000	.....	3,644,000	.....	5,054,500
Total	.....	$19,802,900	.....	$27,935,500	.....	$38,751,400

TABLE NO. 70A.

Probable Cost per Year and per Million Gallons, **Filtered Water from the Hudson River at Poughkeepsie,** *Delivered in New York at City Limits.*

(From partial reconnaissance only.)

For comparison in costs with 250 million gallon plant, see Table No. 8, page 105. The difference is due the economy of large conduit, but would be offset by interest charges.

For plant of 500 gallons, nominal capacity, delivering 400 million gallons per twenty-four hours.

Quantity. Million gallons per year delivered at city limits, 400 x 305=	For 300 feet head at city limits. 146,000 million gallons.		For 135 feet head at city limits.* 146,000 million gallons.	
Construction.				
Cost of pumping plant, including intake works, docks, coal pockets, lands, etc., complete	$9,660,000		$7,648,000	
Cost of filtration works, lands, etc., complete	13,600,000		13,600,000	
Cost of structures for conveyance and storage, Poughkeepsie to city limits	27,937,000		27,935,000	
Park Hill High Pressure Distributing Reservoir	3,696,000		0	
Contribution toward cost of Adirondack Reservoirs, for preventing saltness in drought	1,500,000		1,500,000	
Total cost of works for pumping, filtration and conveying water to the city limits on smallest basis reasonable in absence of surveys.	$56,393,000		$50,633,000	
Operating Expenses.	Total cost per year.	Per mil. gals.	Total cost per year.	Per mil. gals.
Collection and Pumping—				
Interest on entire cost of structures and land at 3% per year	$334,800	$2 30	$274,400	$1 88
Sinking fund to pay off cost in forty years at 3% = 1.326% per year	148,000	1 01	121,300	83
Taxes and special assessments at 1.4% of entire cost	44,600	30	36,600	25
Extraordinary repairs and depreciation at 3% of entire cost	334,800	2 30	274,400	1 88
Coal, labor and supplies, per million gallons raised 1 foot at 3.5 cents	2,300,000	15 75	1,430,800	9 79
Total cost of pumping	$3,162,200	$21 66	$2,137,500	$14 63
Filtering—				
Interest on entire cost of structures and land at 3% per year	$408,000	$2 80	$408,000	$2 80
Sinking fund to pay off cost in forty years at 3% = 1.326% per year	180,000	1 23	180,000	1 23
Taxes and special assessments at 0.4% of entire cost	54,400	37	54,400	37
Extraordinary repairs and depreciation	136,000	93	136,000	93
Maintenance, operation, labor and supplies at $3 per million gallons	438,000	3 00	438,000	3 00
Total cost of filtering	$1,216,400	$8 33	$1,216,400	$8 33
Transportation (Conduits and Terminal Reservoirs)—				
Interest on entire cost of structures and land at 3% per year	$949,000	$6 50	$838,000	$5 75
Sinking fund to pay off cost in forty years at 3% = 1.326% per year	419,500	2 87	370,000	2 54
Taxes and special assessments at 0.2% of entire cost	63,300	43	56,000	38
Extraordinary repairs and depreciation	168,000	1 15	112,000	77
Operating expenses and repairs on a liberal basis	139,000	95	130,000	89
Total cost of transportation, interest and maintenance, etc., Adirondack Reservoir	$1,738,800	$11 90	$1,506,000	$10 33
Total of annual charges, comprising interest, sinking fund, taxes, operation and depreciation	$6,117,400	.....	$4,860,000	.....
Total cost per million gallons of water, from Hudson river at Poughkeepsie, filtered and delivered at New York City limits, including Park Hill Reservoir	.....	$41 60	.....	$33 30
Total, exclusive of sinking fund	5,370,000	36 80	4,188,600	28 70

At these prices city could build works and operate and pay off bonds for entire cost in 40 years, and then own the works "free and clear" and in excellent repair.

*Jerome Park Reservoir is 1.7 miles south of point of delivery at city limits, with high water elevation 131.5 feet above sea.

Appendix No. 11

POSSIBLE SOURCES NORTH OF THE CROTON WATERSHED AND EAST OF THE HUDSON RIVER.

It appears impracticable to secure from the watersheds immediately northwest of the Croton any supply that will flow to the city by gravity and have a head of 300 feet at the city limits, that is sufficiently large to warrant the cost of development. A low level supply, flowing into the New Croton Lake by gravity, can be obtained, but at high cost as compared with the Ten Mile water, and of a size only sufficient for from 7 to 10 years' growth in water consumption by the Boroughs of Manhattan and The Bronx at the present rate of increase. This low level line of supply from Fishkill Creek and other streams warrants a complete survey, if the Ten Mile and Housatonic sources cannot be obtained. These possible sources are outlined on Map No. 15.

A hasty reconnaissance, seeking watersheds available for augmenting the Croton supply by gravity flow, was made late in October and early in November, 1899, under my direction, by Horace Ropes, C. E., through the country east of the Hudson, from Peekskill, as far north as a point east of Albany. I accompanied Mr. Ropes on this as far as Pleasant Valley, nine miles northeast of Poughkeepsie. Two lines of reconnaissance for aqueduct lines were carried along; one about five to eight miles east of the Hudson, called the "high pressure line," intended to deliver water by gravity at the city limits at a hydraulic grade of 300 feet above the sea; the other nearer the river, called the "low pressure line," intended to gather and carry the water at an elevation sufficient to flow by gravity into the enlarged Croton Lake formed by the New Croton Dam.

On the ground, we were disappointed in the hopes that had come from a study of the topographical contour maps of the U. S. Geological Survey. The disappointment came in not finding good dam sites or so good sites for reservoirs of sufficient capacity to develop a high rate of uniform delivery per square mile, as the study of maps had led me to expect.

In planning water supply, abundant opportunity for storage is hardly less important than ample area of watershed, and the Croton, the Ten Mile and the Housatonic above Merwinsville, are better adapted by geological conditions for the development of storage than any other region within practicable distance of New York City.

Northwest of the Croton, particularly along the "high level line," the

streams are short and rapid and the valleys narrow, and while opportunities for developing such a supply as would be needed by a city of 100,000 inhabitants are ample and frequent, the conditions are not favorable to development on the large scale which the present problem demands.

Along the "low level line" the reservoir sites are more hopeful and the drainage areas larger, but to gain these the proposition has to begin with a 16-mile tunnel under the rugged mountain region east of Anthony's Nose, Sugar Loaf, Storm King and Break Neck. To answer definitely about these possibilities a transit line traverse survey with accurate levels and stadia topography and borings at the possible dam sites is needed.

The country being rather difficult and much broken into small streams, does not admit of any very definite statements about this low-level project northwest of the Croton without an instrumental survey; but, as stated above, it appears worthy of investigation, if negotiations for the Ten Mile and Housatonic should fail. These sources could be made to augment the Croton sufficiently to supply the city's real needs for a long time to come, if waste should be vigorously restricted.

Our reconnaissance was based on the geological maps taken into the field, supplementing these with aneroid barometer and hand level, and using an excellent recording barometer stationed at the hotel for correcting the indications of the portable barometer for hourly fluctuations.

The results of the reconnaissance may be briefly set forth as follows: The locations and drainage areas referred to will be found outlined on Map No. 15, but since the scale of that map is too small to show them well, they can be best studied from the topographic sheets of the U. S. Geological Survey.

Reconnaissance on High Pressure Line.

1st. A good site for a high pressure equalizing reservoir is found above Holman's Mills, 2 miles north of Croton Lake and 5 miles east of Peekskill. This at elevation 370 would have area of about 1.25 square miles, and in its upper 10 feet would hold about 2.5 billion gallons, or in upper 20 feet, not far from 4.5 billion gallons. A good dam site is found 250 feet upstream from Holman's grist-mill; good gneiss outcrop on west bank and to bed of stream, east side of valley apparently deeply covered with a "hard pan" formation of unmodified drift or bowlder clay requires borings to make sure of substrata. Dam about 110 feet high at flow line, above the present surface.

2d. Proceeding northerly through the valley by about two miles of shallow tunnel to a point near Shrub Oak, possibly 8.3 square miles of drainage area from Jefferson Valley could be taken in. Dam site not very promising; hardly anything more than a 20 feet earth dam apparently feasible, and storage sufficient to give high rate of yield per square mile apparently difficult to obtain without disturbing houses. Steam found flowing well, considering the long drought, perhaps 3 million gallons per 24 hours, call the possible supply from this 8.3 square miles, roughly, 5 million gallons per 24 hours.

3d. Proceeding thence about 3.5 miles northerly by a fair line for cut and cover aqueduct, the next possibility is Peekskill Hollow Creek with 15.6 square miles of drainage, but on the ground the prospect for a dam site to store any considerable volume is disappointing because of the depth and narrowness of the valley. A dam site further downstream than 1.5 miles southwest of Tompkins' Corner appears impracticable at any reasonable expense. Any dam within reasonable expense could hardly give available storage of more than 800 million gallons in the top 10 feet of reservoir, or the largest possible dam and reservoir would give not more than 2 billion gallons storage; which is 130 million gallons of storage per square mile of watershed. So, even with the opportunity to store part of the flood waters in reservoirs to be constructed at head of drainage area, we could hardly reckon on this Peekskill Hollow Creek contributing more than a safe yield of 500,000 gallons per square mile, or a total of, say, 8 to 10 million gallons per 24 hours.

4th. Continuing north, we would have to go through a tunnel of 7.2 miles in length to the Shenandoah Valley and thence through about 6 miles of cut and cover masonry aqueduct with heavy side-hill work.

No other noteworthy drainage can be found until Fishkill Creek is reached, at a point above which the drainage area is about 50 square miles. The map indicated possibility of a great storage reservoir between Stormville, Beekman and Green Haven, ample for impounding this water, but, on the ground, this did not look at all favorable. A complete survey with accurate levels would be required to demonstrate with certainty what could be done. The surface indications, particularly near the stream itself, call for a dam of great height and of impracticable width, with the bed rock apparently too far below the surface.

There are possibilities of moderate reservoir development on the head waters and in Whaley Pond, but the present data indicate no hope of being able to develop storage such that Fishkill Creek above Stormville could deliver into the high-pressure aqueduct more than at most 25 million gallons per day in extreme drought.

5th. At Lagrangeville the map shows some possibility of securing the drainage from 16 square miles. Large storage at the outlet of this watershed at Lagrangeville appears impracticable on the ground, without more disturbance of residences and railroads, and more expense for dam than the water to be gained warrants. A cursory examination showed no outcrop of ledge near dam site. The map indicates small opportunity to develop storage farther up the valley, so for a first approximation we would not appear justified in assuming at most a safe yield in extreme drought of more than, say, 300,000 gallons per square mile, or say, 5 million gallons per day.

6th. On the head of Sprout Creek there appears to be about 19 square miles of tributary watershed, which may be quite fully developed for storage and made to yield, say, 500,000 gallons per square mile, or at a safe rate, of not more than 12 million gallons per day.

7th. Continuing northward no stream appears available for the high-level aqueduct until the head waters of a branch of Wappinger's Creek are reached, above Pleasant Valley and near Washington Hollow. The area of this watershed is found to be about 4 square miles, with a fair chance of storage. Possibly from this 4 square miles, with liberal expenditure on storage, there could be obtained a safe yield of 3 million gallons per 24 hours.

The aqueduct lines necessary up to this point aggregate 33 miles from the terminal or equalizing reservoir at Holman's Mills, or a total distance of 35 miles north from Croton Lake, and the total safe yield of the streams after liberal expenditure for storage appears to be not more than as follows:

TABLE NO. 71.

Possible Drainage Areas and Safe Yield for High Pressure Water Supply—North of Croton Watershed.

Name of Watershed.	Square Miles of Drainage.	Maximum Safe Yield. Million Gallons per Day.
From Jefferson Valley at Shrub Oak	9	5
Peekskill Hollow Creek at 2.5 miles upstream from Adam's Corner	15.6	10
Fishkill Creek, from near Stormville	50	25
Whortlekill Creek, from near Lagrangeville	16	5
Sprout Creek, from 2 miles northeast of Freedom Plains	19	12
Head of branch Wappinger's Creek, from 2.2 miles east of Pleasant Valley	4	3
Total	114 Square Miles.	60 Million Gallons.

To get this into the Croton Watershed at Holman's Mills requires 33 miles of very expensive aqueduct, of which about 12 miles is tunnel, and about 2 miles is steel-pipe siphon. The remaining 19 miles of this aqueduct line could be of cut-and-cover masonry or more cheaply perhaps of steel pipe, if the line goes only far enough to secure 60 million gallons.

These long tunnels could not be driven smaller than of area about equivalent to a 7-foot circle, which is much larger than needed for hydraulic purposes from the limited watersheds found, and the whole would be unduly expensive for the volume of water to be gained; for this entire addition would be equivalent to only four years' increase of consumption of water by Manhattan and The Bronx alone at the recent rate of increase.

I went no farther myself on this line of reconnaissance, but Mr. Ropes continued it up to a point opposite Albany, looking, however, more for the possibilities of location for an Adirondack aqueduct than for reservoir sites.

At the little village, Hibernia, 39 miles north of Holman's Mills, the main branch of Wappinger's Creek is crossed, but the creek here is about 150 to 175 feet below the hydraulic grade of any possible high-pressure aqueduct, so this valley would have to be crossed by a siphon 1.5 miles long. If a branch line should be run 9 miles up Wappinger's Creek, a suitable grade would be reached for taking in by gravity the flow from about 41.4 square miles, with the map indicating excellent opportunities for storage. This was not followed out on the ground.

Continuing north to Clinton Hollow, forty-two miles from Holman's Mills, on another small branch of Wappinger's Creek the map indicates an excellent reservoir site, which examination on the ground appeared to confirm, but large expenditure would be required for a dam about 75 feet high by 900 feet in length at top, with half its length in drift formation with ledge at uncertain depth, and a fair prospect for ledge at small depth on the other half. The area of this reservoir at 420 feet above sea would be 3.8 square miles. Watershed is 27 square miles, and the reservoir capacity appears possibly sufficient ample for development to 900,000 gallons per square mile per day, or for the 23 miles of net land drainage, say, at most 21 million gallons per 24 hours.

Continuing to the 58th mile at Elizabeth at the Rocliff Jansen Kill, the valley is nearly 200 feet below hydraulic grade, and the water cannot be taken into any gravity supply without a long branch line. There is at present no U. S. contour map showing this drainage area, but other maps indicate perhaps 75 or 100 square miles of watershed, with possibilities of dam sites in its deep gorge some miles to the east, by which water could be made to enter the high level aqueduct. This was not explored.

The total available high pressure sources found in going this long distance north from the Croton are given below.

TABLE No. 72.

Possible Drainage Areas for High Level Water Supply North of Croton Watershed.

	Square Miles Drainage Area.	Maximum Safe Yield. Million Gallons per Day.
By previous summary, Table No. 71.....	114	Yielding possibly..... 60
At head of Wappinger's Creek.....	41	" 25
Clinton Hollow.....	27	" 21
Rocliff Jansen.....	75–100	Not explored—perhaps 40
Total.....	257 Square Miles.	146 Million Gallons.

The aggregate yield for all this length of sixty miles of aqueduct with numerous dams would be less than could be diverted into the Croton Watershed at equal elevation at Pawling, without any aqueduct, by one dam at Webatuck.

In continuing the reconnaissance for the Adirondack aqueduct farther north, Mr. Ropes noted that the larger streams of Claverack and Kinderhook Creek were too low to be diverted into a high pressure gravity aqueduct unless branches should be run easterly up toward their head waters. As these were off the limits of the U. S. Geol. contour maps, after a brief study of railroad profiles for their elevations further reconnaissance for small watersheds lying north of the Croton capable of giving a gravity high pressure supply was discontinued.

Reconnaissance for Low Pressure Supply—Northwest of Croton.

Failing to find adequate supply at reasonable cost that would flow by gravity to an elevation of 300 feet above sea at the city limits, the broader possibilities of a low level line delivering into Croton Lake remain to be considered. An apparently possible aqueduct line had been sketched on the maps before going out on this reconnaissance, and this was examined on the ground at a few critical points to get some idea of the difficulties of construction.

North from Croton Lake the line immediately runs against the great natural obstacles presented by the range of rugged hills or mountains east of Peekskill, and the detour along the Hudson would involve so much extra length, and incur so much very heavy steep hillside work that the only practicable method appeared to be to start north with a tunnel 16 miles long, beginning at the northern arm of the new Croton Lake 1.5 miles east of Dickerson's Hill and running to a point on Clove Creek about 2 miles south of Fishkill.

Watersheds Available.

1st. The first stream crossed is Peekskill Hollow Creek, where, near the Village of Putnam Valley, about 30 square miles of drainage could be diverted at a point 5 miles up from the outlet of the tunnel. No exploration for dam sites was made so far down on Peekskill Creek, but what was seen farther up, together with what is shown on the map, leads me to expect considerable storage at reasonable cost. Perhaps with Oscawanna Lake, with some dams farther up the valley, and with such dam as could be built near Putnam Village, sufficient storage could be secured to give a safe yield of 500,000 gallons per square mile, or from the 30 square miles of watershed there could perhaps be obtained a safe yield of 15 million gallons per day.

2d. Two miles further up stream, Sprout Brook or Canopus Creek is crossed, having 8 miles of drainage. This could, perhaps, be dammed and

let into the tunnel down the shaft. No local examination was made of dam and reservoir site. To make the most of these possibilties we will assume a safe yield of 300,000 gallons per square mile, this affording a total of 2.5 million gallons per day.

3d. Five miles further north, Sand Spring Brook, with 5.5 miles of drainage could probably also be dammed and let in, but no examination was made on the ground for dam and reservoir sites. Assume provisionally a safe yield of 400,000 gallons per mile, or from the 5.5 square miles a total of at most 2 million gallons per day.

4th. At upstream portal of tunnel, 13.6 square miles draining northward down Clove Brook could probably be let in. Fair opportunity for storage, say 600,000 gallons per mile, or a safe yield of at most 8 million gallons per day.

We have so far gone 16 miles through an expensive tunnel with little hope of thus far obtaining a safe yield of over 27.5 million gallons per day.

Thence a gravity line would proceed northwest by cut and cover aqueduct to Brinkerhoff.

Fishkill Creek Source.

5th. Here the aqueduct would tap Fishkill Creek, draining 158 square miles, and with promise of a reservoir site of area ample to store and develop a very large proportion of the total run-off.

Hasty examination of the site showed that borings would be needed before anything definite would be known about nearness of ledge to surface. The surface indications are fairly favorable and indicate that a long, low earth dam with masonry core would serve to impound a very large quantity, but apparently there would be much shallow flowage, and there is a possibility of some leaky gravel subsoil at the dam site. A stadia survey with accurate levels would be needed for definite knowledge of the storage and shallow flowage, and borings would be needed to test the quality of the dam site. So far as may be judged from the topographical maps and the railroad levels and general appearances it seems fair to assume a possibility for storage sufficient to give 11,000 million gallons available capacity with 10 feet average depth over the very broad area of 5.5 square miles, or 16 billions if 15 feet average depth of draft is found practicable, and perhaps more. For a first approximation, we will assume the storage on this entire watershed to average 100 million gallons per square mile. The total drainage area above this point is not far from 158 square miles. The total yield depends on the quality

of the dam site as to be revealed by borings and accurate surveys and whether the geological study shows the substrata impervious. The area of storage basin is ample to develop a safe yield of 600,000 gallons per square mile per day, and perhaps 700,000, and there is thus good hope of securing from Fishkill Creek a safe yield of 100 million gallons per day. **Or, in going this distance of about 20 miles from Croton Lake we find excellent promise of securing the drainage of 220 square miles of watershed with a safe yield of 180 million gallons per day.** If time permitted I would have confirmed this hasty reconnaissance by an instrumental survey.

6th. Wappinger's Creek, the next considerable stream, is of such importance to the extensive print works of Garner & Co. for wash water, that the damages for diverting any considerable portion might prove much more burdensome than a diversion of water used only for power. The stream is so deeply eroded that its waters could not be diverted without following its bank either with a cut and cover aqueduct, or its equivalent, a long distance up the stream to possibly 1 mile above Hibernia, or, on the other hand, building a high expensive dam perhaps near Pleasant Valley, or possibly much further upstream, involving much railroad re-location. Very likely it might be found by accurate surveys that 139 square miles of watershed is available, and that this could be relied upon for 50 or 60 million gallons. We have thus, in a distance of 33 miles, or adding the 13 miles of branch lines up Wappinger's Creek, making a total distance of 46 miles from the Croton Lake, found good probability of securing 180 million gallons of water per day, sufficient for twelve or fifteen years' increase in consumption of Manhattan and The Bronx only.

To sum up; from this crude reconnaissance I did not find the low pressure line as just described, presenting any attractiveness in comparison with the Ten Mile, but believe it will warrant three months work of a topographical party well equipped for accurate explorations, because if the securing of water rights upon the Ten Mile and the Housatonic at a reasonable price is slow in negotiation, or ultimately fails, then it is important that no time should have been lost in the planning and readiness for securing the next best addition and for weighing the advantages and defects of this watershed north of the Croton against those for watersheds west of the Hudson river.

Appendix No. 12.

A SUPPLY FROM THE ADIRONDACKS.

The Adirondack Mountain region has been so often mentioned as the ultimate source to which New York City must look for pure water, much as Winnipiseogee Lake was formerly often mentioned as the necessary future supply for Boston, that it appeared proper to give attention to its possibilities, particularly since some years ago (1884) Mr. J. T. Fanning, an engineer of much prominence in hydraulic matters, prepared a report with estimates that a supply of 500 million gallons could be brought to New York from the Adirondacks for about fifty-five million dollars, and that this aqueduct or canal could be readily enlarged by building its banks higher, so as to supply upward of 1,000 million gallons per day.

I find no possibility that the necessary structures could be built and the necessary rights acquired without adding 75 per cent. to that estimate of cost. The excellent contour maps of the U. S. Geological Survey, which now cover the greater part of the distance, were not available at the time of Mr. Fanning's estimate, and I do not understand that he made any instrumental survey over the line. The extent and value of the water-power developments along the upper Hudson have increased largely since the date of Mr. Fanning's estimate (Mr. Rafter says 13,000 water horse-power were in use from the Hudson in 1882; and that this use had increased to 55,000 horse-power in 1898), and other very large developments are now being promoted. It is also but fair to state that Mr. Fanning's proposition did not take the water quite so far up the stream and did not deliver it at so great pressure in New York as the plans that I have prepared.

A location for an Adirondack aqueduct on the high lands east of the Hudson was kept in view during the reconnaissance for the high-pressure aqueduct line northward from the Croton, described in Appendix No. 11, and also in the reconnaissance for the low-pressure line which had filtered water from the Hudson at Poughkeepsie in view. For this reason, Mr. Ropes' reconnaissance for aqueduct location was carried farther north, by driving along the highways and briefly examining the difficult locations as far as a point opposite to Albany. In the latter part of October, 1899, I made a brief examination of the ground for 10 or 12 miles at the upstream end, or from Hadley to the great bend in the river a few miles below Palmer's Falls,

and particularly at the point which a study of the maps and a little previous personal experience in this region had indicated as *the nearest* favorable point for diverting the water of the upper Hudson into an aqueduct.

(In order that basis of my Adirondack estimate may be clearly understood, it is necessary to state that in any preliminary reconnaissance there is a considerable margin of uncertainty beyond the numerous little extras which in most engineering experience are found to add about ten per cent. to the total of the main items.

In the case of the Housatonic and Ten Mile, it was my effort that this margin of uncertainty should be included as adding to the cost, but in the Adirondack estimate there was rather a disposition to look at the cost in as favorable a way as could be fully justified, though with no intention of underestimating any item, therefore the nearest favorable point was taken for the point of diversion.)

I had no time for exploration of the individual reservoir sites, excepting a brief view of the proposed Hadley dam site, and it is possible that instead of merely damming the river at the head of the Corinth Falls, and at that point diverting the combined waters of the branches converging below Hadley, that a favorable plan could be worked out for reserving and developing, solely for the supply of New York, one or more of the enormous reservoir sites with which nature has favored this region, cutting brush wood and removing the stumps from this basin that is to store water for drinking purposes and municipal supply, and then develop by cheaper methods another series of reservoirs on the remaining branches of the Hudson for the purpose of storing flood waters and thus providing compensation water that, in the dry season, would take the place of the water diverted and thereby avoid nearly all claim for water damages on the lower river.

The Corinth site appears to serve all the purposes of the present investigation and sets forth the Adirondack possibilities in a favorable light.

Safe Yield of Adirondack Sources.

The large drainage area above the point thus selected as the nearest favorable location for the diversion works is more than 2,650 square miles, and the well-known lake systems of the Adirondacks indicate that this area could be made to furnish an ample source for the 1,000 million gallon supply stated as feasible by Mr. Fanning, but reference was also had to the statements in the water power volume of the U. S. Census of 1880 (The Benedict Surveys) and particularly to the recent investigations on the yield and on opportunities for storage to increase the yield made in the Adirondack region by Mr. George D. Rafter, and set forth in the Report of the State Engineer of New York to the General Assembly for the year 1895, pages 89 to 195; also to the Report on the Water Resources of the State of New York, Bulletins Nos. 24 and 25 of the U. S. Geological Survey, page 134 of No. 25, etc., which, in another form, give the result of the same studies.

Mr. Rafter, from his later studies, confirms what Mr. Fanning had said and reports that very remarkable facilities exist for constructing large storage reservoirs in the Adirondacks at low cost, and that these reservoirs, above the point that I have proposed for diversion, can be readily made large enough to so supplement the natural flow that a uniform flow of from 3,000 to 3,600 cubic feet per second or from 1,900 to 2,280 million gallons per 24 hours could be maintained at Glens Falls as an average for the driest month of an unusually dry year like 1891. The proposed diversion of 1,000 million gallons per 24 hours, the largest volume considered for conveyance to New York, amounts to only half of this quantity.

He further calls attention to the remarkably favorable site found at Hadley Bridge, by which 4 billion cubic feet or 30 billion gallons could be stored at the foot of a watershed of 1,642 square miles, with a very cheap stone dam, in a deep gorge. I visited this site and it certainly looks very favorable for an enormous storage at low cost. Mr. Rafter's reports to the State Engineer gives estimates upon other very ample reservoir sites.

LOW-WATER FLOW OF THE UPPER HUDSON RIVER.

The low-water flow for a month at a time under the present conditions of storage, although aided by outflow from lumberman's dams, may sometimes, as we gather from Mr. Rafter's papers, run as low as 600 cubic feet per second, which is only about 400 million gallons per 24 hours at the proposed point of diversion.

The U. S. Census Report on Water-power, 1880, Volume 16, page 346, quotes Mr. Warren Curtis, the designer of the extensive plant of the Hudson River Pulp and Paper Company, as stating that the flow at Palmer's Falls, for one or two months in a year, sometimes falls as low as 200 to 250 cubic feet per second, which is only about 150 million gallons per 24 hours, and incredibly small, coming from so large a watershed.

While there is no doubt that the present rate of summer flow is small, all authorities concur in the very remarkable possibilities for large storage at low cost along its head waters, and without the expense and delay of further study, we appear entirely safe in the assumption made above, that from this 2,650 square miles of watershed, there can be diverted in the driest years from 800 to 1,000 million gallons per day with a safe margin left at the State dam for feeding the Champlain Canal; also for supplying the needs of all present factories with ample wash water and beyond this a fair probability that the full present use of water-power at all Hudson river mill sites can be maintained at its present magnitude by compensation water from future flood storage in these reservoir.

PRACTICABILITY OF THE ADIRONDACK SOURCE.

While the providing of New York City with a domestic water supply from the Adirondacks may be found impracticable as a business proposition, in competition with the cheaper sources near at hand or because of restrictions on the city's borrowing capacity, *it was, as a matter of hydraulic engineering found to be entirely practicable to divert the required quantity of water from the Hudson below Sacondaga Creek, in the Town of Corinth, near where the iron highway bridge from Hadley to Corinth crosses the river, just above the head of Palmer's Falls, at an elevation of 545 feet above the sea, and carry it by gravity to New York City and deliver it at an elevation of 300 feet above the sea at the city limits.*

The many millions required to carry out this Adirondack project look like a moderate expenditure in comparison with the expenditures recently proposed for the reorganization and betterment of the water supply of London.

So far as was developed by our brief reconnaissance in the field over about three-fourths of the entire distance, with the contour maps of the Geological Survey in hand, on which certain apparently possible locations had been previously traced, there are no obstacles greater than frequently have to be overcome in railroad, canal and aqueduct location, and it was found almost as cheap to so locate the line as to deliver the water at 300 feet elevation at the city limits as to deliver it at the level of the Croton Aqueduct.

Best Form of Conduit from the Adirondacks.

To obtain the cheapest possible aqueduct, the study began with a line of open canal, similar to the ordinary water-power canal and not unlike that proposed by Mr. Fanning, except that a more economical type of retaining-wall was designed, which, although cheaper, has been well tested by experience.

To my surprise, by a comparative study of cost and delivery of various sections, taking the steep side-hill slopes into consideration and considering also that the velocity in a long open canal in the severe climate in which this canal would have to be operated, is limited to a mean velocity for the entire section of not exceeding about 2.25 feet per second, or a velocity of say, 2.50 feet per second at the surface near the middle of the stream, by the tendency of the ice to break up and form gorges under certain conditions of temperature, snow and wind, it was found that with the rate of fall per mile available the " cut-and-cover " aqueduct sections shown in Appendix No. 4 Figs. 60–

65, with their greater smoothness of wetted permieter and greater hydraulic efficiency and the much higher mean velocities permissible without scour or danger, *were decidedly more economical in first cost of construction than the cheapest possible open canal.*

Considerable personal experience with anchor ice also convinces me that this masonry aqueduct, covered with 5 feet of earth, drawing from a proper intake, would be much safer against interruption by anchor ice than any possible open canal. This is an important consideration, since municipal water supply is far more exacting as to safety against interruption than supply for water-power.

My views on the limiting velocity in a long open canal for water supply, which led to the abandonment of the open canal with masonry walls in all side-hill work in favor of the cut and cover aqueduct, may be thought too conservative, but they are grounded upon my practical experience for a good many winters on a large water-power canal, where there was ample opportunity to study the breaking up of ice and the clogging by a heavy snow-fall followed by extreme cold, and by some experience with another water-power canal about 2 miles long, carrying at the rate of about 1,000 million gallons per 24 hours, upon which I have had occasion to study ice movement, and found that a mean velocity of a little above 2 feet per second was safe.

Allowance for the loss of area due to ice 2 feet thick and for the friction of the increased wetted perimeter, adds much to the size and cost of a long open canal.

Cut-and-cover aqueduct was therefore used wherever proper ground for it could be found, because this appears to be the cheapest and best type of aqueduct yet known, except that an open canal without side walls is the cheapest through a level plain or near the bottom of a valley, or parallel with the drainage of the country, as from Pawling to Milltown Bridge. This aqueduct line from the Adirondacks, either by the high pressure line or the low-pressure line runs almost continuously, across the drainage of the country, and there is hardly five miles in the whole 180 where this cheap type of open canal could be used.

BASIS OF ESTIMATING COST OF ADIRONDACK PROJECT.

Diagram No. 66 was, therefore, used as the basis of the following, roughly approximate estimates of cost. In special structures for crossing ravines, in assumptions as to requirements of tunnel lining, in regions where some schist is found in the outcrop, in estimates for damages for land and for water diverted, and in cost of reservoirs, these figures are purposely on a little lower scale of costs than my estimates for the Ten Mile development or for the proposed aqueduct lines from Sodom to the city limits, in other words, the probable cost of the uncertain features was not so exaggerated as for the Pawling Canal, the new highways in Dover, Kent, etc., or as for the water diversion damages on the Housatonic.

In the figures of the following table the tunnels are assumed to be built of full-size from the start. The estimate was cheapened by omitting the arch from the tunnel so far as appeared justifiable from notes on the outcrops of

ledges and noting where the country rock was granite and where schist. Above Pleasant Valley the country rock does not look favorable for low-cost tunnels.

Possible Aqueduct Location West of Hudson.

For a complete study of the aqueduct possibilities a line on the west of the Hudson river should also be explored. A study of the maps indicates that it could not be materially shorter than the one which we reconnoitered and that the only hope of saving would be in the possibility of finding the cross valleys less difficult, the tunnels shorter or that a smaller amount of steel pipe could be used. From such incomplete maps as exist, I infer that the proportion of steel pipe and of difficult work would be larger. There is no possibility that the saving could be enough to materially affect the feasibility of the project or its cost as a whole.

Adirondack Storage Reservoirs.

Proper data for the cost of such work as would be required to prepare the basins so the water will be at all times fit for domestic supply is lacking and in the brief time at my disposal, I have, of course, had no time to do much more regarding storage than to utilize the description and estimates given in Mr. Rafter's very interesting report to the State Engineer in 1895.

Mr. Rafter then having water-power and navigation in view, estimated that to develop a safe yield of 3,000 to 3,600 cubic feet per second at Glens Falls, or 4,500 cubic feet at Mechanicsville, in a year of unusual drought, but not so severe as may come two or three times in a century, would require a reservoir capacity of 45,593 million cubic feet, and that this could be secured and constructed at an average cost of about $60 or $70 per million cubic feet, which is a reservoir cost of less than $10 per million gallons of capacity. On page 172 of the New York State Engineer's Report for 1895, Mr. Rafter estimates the entire cost of the proposed Upper Hudson River Reservoir System at about 2,600,000 as prepared for water-power purposes.

Mr. Rafter estimates, page 175, that with the exceptionally advantageous rock formation existing at Hadley Bridge, the proposed Hadley Reservoir, holding available over 30 billion gallons at the outlet of 1,642 square miles of watershed, could be constructed for $360,000, including relocation of railroad. For purposes of development and protection for domestic water supply the

value of the Pulp Mill near by would have to be added, also the expenditure necessary to cut, grub out and burn the bushes and stumps on the reservoir margins.

The cheapest of the Croton Reservoirs has cost about ten times this rate per million gallons and, in general, the superior permanence of dam and extra cost of cutting out the stumps and brush, etc., from the bottom of a reservoir for domestic water supply, will make it cost much more than an ordinary reservoir for water-power or lumbering, on which more chances of failure may properly be taken and on which no expensive work for avoiding bad tastes in waters need be considered.

If, at a venture, we must set down some figure for the cost of the necessary dams and reservoirs for supplying water to fill the several sizes of aqueduct in the following table, it may be, so far as I now have definite knowledge, anywhere from three million to five million dollars for the 500 million gallon supply, and proportionally more for the 1,000 million gallons per day; and this cost for storage reservoirs would be still more if "compensation reservoirs" are provided, in the effort to lessen or avoid the payment of heavy damages on account of injury to the water-power at the many valuable mills on the stream below.

"Compensation reservoir" is a term used to denote a reservoir built for the purpose of providing an equivalent of the water diverted—compensation by water instead of by cash. For example, on the Adirondack watershed the water of one large tributary might be diverted to New York without injury to the water power of any mill owner on the stream below, if the summer or low water yield of another tributary should be augmented by the discharge from a compensation reservoir of a volume of water equal to that diverted; this compensation water being obtained by the storage of spring flood water which now runs to waste.

The very large total capacity of the storage reservoirs that can be cheaply constructed in this Adirondack watershed may make it possible to provide reservoirs large enough to secure a flow sufficient for New York's largest demands, and at the same time leave the summer flow in the river below the point of diversion nearly as large as now.

In the earliest water works for Boston some compensation reservoirs were provided, and this method of compensation is understood to be common abroad, but has seldom been followed in the United States.

Water Diversion Damages on Upper Hudson.

This question is at the outset complicated by the considerations that if large impounding reservoirs were built by the city or the levels of some of the large Adirondack lakes raised and controlled by dams and sluices for the storage of the spring flood waters and if no more water were drawn into the aqueduct than the measured outflow from these new reservoirs less their present summer flow, no taking away of benefits heretofore enjoyed would be suffered by the mill-owner, but a gain would come to him from having the flood depth reduced. As another method of avoiding damages the entire discharge from one tributary, as from the proposed dam at Hadley

Bridge, or perhaps preferably from the Schroon, might be diverted into the aqueduct without injury to the mills below, if compensation water were discharged meanwhile from reservoirs on other tributaries built by the city for this purpose of keeping up the present rate of discharge in dry months.

It is rare almost to the point of being unknown, for a municipality to divert water without having to pay heavy damages, and the chances are that with troublesome adverse rights acquired in the stream by the lumbermen, the ideal condition of compensation *in kind* instead of in cash, considered above, could not be realized.

To make our estimate a safe one we must, therefore, estimate on such procedure as commonly prevails and assume payment for water diverted as if the mills had proposed to develop the Adirondack reservoirs for their own benefit.

First, at Hadley, any such development as required, would necessitate the purchase and removal of the extensive Pulp Mill, unless the point of diversion were moved upstream by extending the aqueduct at large expense.

Next, at Palmer's Falls, the enormous plant of the International Paper Company, one of the largest in the world, would suffer material injury, unless the plan of compensation reservoirs prevailed.

Next, the proposed water-power development near where the river breaks through the Luzerne Mountains would have its prospects for power injured, for it is understood to be the hope of all these mill-site owners that the storage reservoir sites of the Upper river are eventually to be developed, to aid navigation, etc., and, incidentally, to their own great benefit for water-power, and it would doubtless be claimed that this opportunity for developing storage on the head waters, instead of being one for the Legislature to give to the city, was a natural advantage and right appertaining to their several mill sites which could not be extinguished without compensation.

AMOUNT OF HUDSON WATER-POWER DESTROYED.

Developed Power:

From Mr. Rafter's Table No. 12, page 150, Rept. State Engineer, 1895, it appears that of the total fall of 455 feet below the point of diversion, about 310 feet is already in use.

Undeveloped Power:

A company is being promoted at the present time to build a very high dam a few miles below Palmer's Falls, near where the river breaks through

the Luzerne Mountains, and utilize perhaps 90 feet out of the 140 feet fall now unused between the pond of the State Feeder Dam, above Glens Falls, and the foot of Palmer's Falls, or this has so been stated to me as the available fall by one interested in the project.

Computation of Amount of Power that the Water Proposed to be Diverted Would Produce if Used at the Present and Prospective Mill Sites on the Hudson River Below Palmer's Falls.

$$\text{Net horse-power} = \frac{\text{million gallons} \times 8.33 \text{ lbs. per gal.} \times 310 \text{ ft. fall} \times 74\% \text{ efficiency.}}{86{,}400 \text{ seconds per 24 hours} \times 550 \text{ foot pounds.}}$$

MILLION GALLONS PER DAY TO BE DIVERTED.	200.	400.	800.
Developed Sites—			
Net horse-power this quantity of water would produce at mill sites already developed, if there were sufficient turbines (24 hour power)..	8,000	16,000	32,000
Undeveloped Sites—			
Net horse-power this quantity would produce on 90 feet fall.....	2,320	4,640	9,280

Mr. Rafter computes that the present diversion of water to feed the Champlain Canal at Glens Falls, amounting to from 255 to 419 cubic feet per second, equivalent to from 165 to 270 million gallons per 24 hours, deprives the mill-owners at Glens Falls and below of water-power that has, at a moderate estimate, a market value of $46,830 per year, and is equivalent to about 4,683 horse-power, 24 hours per day, and (p. 159, Report of State Engineer, 1895), that if the 24-hour water-power had to be replaced by steam power at the several sites, the extra cost to the mills for permanent 24-hour power would be at least $40 per horse-power per year.

$10 per year is 5 per cent. on $200.
$40 " " " 5 " " " $800.

It is rare that actual settlements including legal expense, for developed water-power taken away in connection with the development of a public water-supply are less than $400 per horse-power of 12-hour power; for small mills in populous districts the awards are sometimes much more than this.

VALUE OF THIS DIVERTED POWER.

If, considering that this diversion is not felt during two-thirds of the year, and that there are not yet turbines at some of the dams to utilize the whole summer flow, and making some allowance for possibility of compensation water, we call this worth on the average $200 per horse-power for the developed and half this or $100 per horse-power for the undeveloped power, we arrive at—

For	200	million	gallons	per	day	diverted,	damages.....	$1,880,000
	500	"	"	"	"	"	"	4,700,000
	1,000	"	"	"	"	"	"	9,400,000

If the reasonable method of payment, in part, at least, by the construction of compensation reservoirs did not prevail, and if the damages were to be figured on the steam-substitution basis the damages might be very much larger than computed above, perhaps double that amount.

As showing the upper limit toward which the damages for that part of the power already in use might tend under the pleading of ingenious counsel, Mr. Rafter's statement might be quoted from the report of the State Engineer already referred to, where he estimates that the extra cost of steam power over water power in this particular locality, for the 24 hour power required by pulp and paper mills, is $40 per horse-power per year. If the season of damage of the low-water period be assumed to continue during half the year the annual cost of substitution on this basis would be somewhat more than $20 per net horse-power per year, which capitalized at 4 per cent. gives a value of $500 for each horse-power diverted from turbines already in use.

There have been various instances of diversion damages in the case of water taken for municipal supply costing, together with the attendant expenses of trial, counsel and witness fees, even more than this, therefore I do not consider that smaller values can fairly be used in our rough preliminary estimate than those adopted in Table No. 73, notwithstanding that this power is not yet fully developed and that in all cases the diversion would cause real injury during only a fraction of the year.

The final estimate showing the probable cost of water rights, collection, storage and transportation of this Adirondack water is given in Table No. 9 of the main report.

LOW-PRESSURE AQUEDUCT LINE.

A study for an aqueduct line at a lower level was made over part of the course, but the tunnels east of Peekskill were longer than for the high pressure and the saving in cost due to the greater declivity did not appear large enough to recommend that line, and if the Adirondack project is to be entertained, I consider it certain that the delivery at the city limits should be established at an elevation not less than 300 feet above the sea, the same as proposed for the Ten Mile and Housatonic plan.

CONCLUSIONS ON ADIRONDACK SOURCE.

Summing up the Adirondack proposition, it can be set down as a certainty that an abundant supply of most excellent water for New York could be obtained from this source, but this source cannot compete successfully with several other sources because of the great expenditure necessary before even the first 200 million gallons could be delivered.

TABLE No. 73.

ADIRONDACK PROJECT.

Estimated Cost of Conduits, from Head of Palmer's Falls in Corinth, N. Y., to City Limits (Woodlawn) via "High Line," Mainly East of the Hudson River. Distance, 188 Miles (Steel Pipe, 56 Miles; Cut and Cover Masonry Aqueduct, 101 Miles; Tunnel, 31 Miles).

Assumptions—Hyd. Grade Elev.—
Intake..... 545 ft.
Terminus..... 300 "
Slopes—
101 miles Cut and Cover, at 0.9 per mile 90.9 ft.
31 m. tunnel, 1.0 per mile ..... 31.0 "
56 m. pipe, 2.0 per mile ..... 112.0 "
Loss at Holman's Reservoir..... 10.0 "
Minor losses of head..... 1.1 "
Total head ..... 245.0 ft.

	Daily Average Supply, 200 Million Gallons. Capacity of Aqueduct (clean), 250 Million Gallons. Average Inside Diameter of Cut and Cover Aqueduct, 11.5 Feet.		Daily Average Supply, 400 Million Gallons. Capacity of Aqueduct (clean), 500 Million Gallons. Average Inside Diameter of Cut and Cover Aqueduct, 15.1 Feet.		Daily Average Supply, 800 Million Gallons. Capacity of Aqueduct (clean), 1,000 Million Gallons. Average Inside Diameter Cut and Cover Aqueduct, 19.8 Feet.	
	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.	Cost per Unit.	Total Cost.
101 miles Cut and Cover aqueduct..	$157,000	$15,857,000	$215,000	$21,715,000	$306,000	$30,906,000
16 miles arch lined tunnel	348,000	5,568,000	474,000	7,584,000	679,000	10,864,000
15 miles side lined tunnel	284,000	4,260,000	390,000	5,850,000	538,000	8,070,000
44 miles steel pipe $\frac{7}{16}$-in. for 140-ft. head	Diam., 8 ft. 6 in. 160,000	7,040,000	Diam., 8 ft. 6 in. 160,000		Diam., 8 ft. 11 in. 176,000	
8 miles steel pipe $\frac{8}{16}$-in. for 190-ft. head	177,000	1,416,000	177,000	9,051,500	188,000	9,878,600
3 miles steel pipe $\frac{11}{16}$-in. for 240-ft. head	198,500	595,500	198,500		210,200	
1 mile steel pipe for 400-ft. head	Diam., 7 ft. 3 in. 212,000	424,000	Diam., 7 ft. 9 in. 232,000	464,000	Diam., 7 ft. 9 in. 232,000	464,000
Present value of additional future pipes discounted at 3 per cent. per annum—						
2d line of pipe deferred	6½ yrs.	7,470,000	6½ yrs.	7,470,000	7⅛ yrs.	7,915,000
3d line of pipe deferred	.....	.....	13 "	6,164,000	15 "	6,341,000
4th line of pipe deferred	.....	.....	20 "	5,012,000	23 "	5,006,000
5th line of pipe deferred	.....	.....	.....	.....	30.5 "	4,010,000
6th line of pipe deferred	.....	.....	.....	.....	38 "	3,213,000
7th line of pipe deferred	.....	.....	.....	.....	46 "	2,536,000
2d line of pipe (400 ft.) deferred	9 yrs.	162,500	10.5 yrs.	340,200	10.5 "	340,200
3d line of pipe (400 ft.) deferred	.....	.....	21 "	124,700	21 "	249,500

4th line of pipe (400 ft.) deferred	.....	.....	.....	.....	32 "	180,200
5th line of pipe (400 ft.) deferred	.....	.....	.....	.....	43 "	130,200
4,600 linear feet shaft excavation	$125	690,000	$160	736,000	$200	920,000
34 siphon chambers	10,500	357,000	14,000	476,000	21,000	714,000
Bridges, culverts, manholes, blow-offs, etc., 157 miles	15,000	2,355,000	21,000	3,297,000	30,000	4,710,000
Right of way 101 m., C. and C., 100 ft. wide	3,000	303,000	3,000	303,000	3,000	303,000
Right of way 31 m., tunnel, 50 ft. wide and shaft	.....	92,000	.....	95,700	.....	100,800
Right of way 56 m., pipe line	60 ft. wide 1,800	100,800	90 ft. wide 2,700	151,200	150 ft. wide 4,500	252,000
Diversion dam and reservoir at Palmer's Falls	.....	100,000	.....	100,000	.....	100,000
Equalizing reservoir and dam at Holman's Mills	.....	1,550,000	.....	1,550,000	.....	1,550,000
	.....	$48,340,800	.....	$70,484,300	.....	$98,753,500
Add 15 per cent. for supervision and contingencies	.....	7,251,100	.....	10,572,600	.....	14,813,000
Total for aqueducts, etc., exclusive of storage and water rights on the upper Hudson river	.....	$55,591,900	.....	$81,056,900	.....	$113,566,000
Damages account diversion of water, say	.....	2,000,000	.....	5,000,000	.....	10,000,000
Storage works on upper Hudson river, say	.....	2,000,000	.....	3,500,000	.....	5,000,000
Terminal reservoir at Park Hill	.....	3,696,000	.....	3,696,000	.....	5,542,000
Total cost of water rights and structures for storage and conveyance of water, Palmer's Falls to city limits, at elev. 300 above sea	....	$63,287,900	.....	$93,253,000	.....	$131,108,000

For estimate of cost per million gallons of water delivered see Table No. 10.

A WATER SUPPLY TO NEW YORK CITY FROM LAKE GEORGE.

Lake George has also been often suggested as a source of supply for New York City. This is an utterly hopeless source, because its drainage area, and, therefore, the volume of water that can be drawn off, is much too small. The watershed of Lake George, according to the Census Report of 1880 on Water-power, is 238 square miles. This is confirmed by tracing and planimetering the area on the more recent maps of the Geological Survey, from which the drainage area is found to be 235 square miles. Its area of water surface by planimeter on map is 45 square miles. Raising the dam at its outlet so as to secure a rise and fall of 7 feet, would give a storage of 243,000,000 gallons per square mile, and thus by the Sudbury Diagram, Figure 49, afford 800,000 gallons per square mile per day from its 190 square miles of net land surface, or a total yield in a dry year of 152 million gallons per day.

This is too small a quantity to warrant going so far to seek. Moreover, the elevation is inadequate to deliver it in New York through the necessarily long aqueduct line at an elevation sufficient to avoid pumping. The elevation of Lake George is 323 feet above the sea, or 222 feet lower than the Hudson at the point of diversion proposed in the Adirondack plan. Moreover, an aqueduct line to New York from Lake George would be 7 miles longer than an aqueduct from the Adirondack watershed at Corinth.

The Census Report on Water-power, 1880, Volume 16, Part 1, page 406, estimates minimum flow as now controlled by the Lake George Dam as ordinarily 330 cubic feet per second.* This is equivalent to 215 million gallons per 24 hours; and the average flow for the year is estimated by the census report at 450 cubic feet per second, equivalent to 290 million gallons per 24 hours.

Lake George should be permanently dismissed from consideration as a source of supply for New York City as being beyond even the most remote possibility.

The Great Lakes.

The simplest measurement on a map like Figure 1, shows that any conduit to Lake Ontario would be very much longer than the conduit to the outlet of the Adirondacks, and as the main expense of the Adirondack

* This estimate is larger than that stated above because of being on the basis of water power and less conservative in regard to extreme drought than is required in estimating the safe yield for purposes of public water supply.

project is in the conduit, any Great Lakes project is far outside the limits of reasonable propositions. Moreover, while the Adirondack water is taken at an elevation 540 above sea, water from Lake Ontario would require enormous expenditure for overcoming friction of the long line of aqueduct and for maintaining the head at the point of delivery.

Appendix No. 13.

SOURCES IN SOUTHERN NEW YORK, WEST OF THE HUDSON.

The Ramapo River and the Delaware River Tributaries.

Unfortunately, the excellent topographical maps of the U. S. Geological Survey, made in co-operation with the State of New York, as yet cover a very small part of this region, and any work of reconnaissance for water supply on a large scale is very slow without them; indeed, it would be profitable for New York City to add double to the State's small annual appropriation for these maps, in order to have them complete before January 1, 1901, for all regions in which water can be reasonably sought.

Obviously, I was unable to personally explore all of the above possibilities in the brief time at my disposal, but that we might be certain of the possibilities in a general way, detailed one of my assistants to obtain the elevations along the railroad lines and follow along the important streams looking for sites for storage reservoirs. He spent about two weeks in a diligent examination along the Mahwah, Ramapo, Wallkill, Neversink, Basher's Kill and other tributaries of the Delaware; provided with the best county and township maps that could be obtained, driving over the roads nearest the streams and making diligent enquiry from those familiar with the country, and examining the most promising points.

Obviously, so brief an examination could not be satisfactory and the region perhaps merits some further reconnaissance aided by some hasty levels and stadia work, more for the sake of completeness and to be able to say that all possible sources have been explored than for following up any hopeful indications.

Personally, from a study of the maps, the geological reports, the railroad levels and such information as this brief exploration developed, I feel already well assured that this region offers no source that can possibly equal the Ten Mile or several of the other sources explored.

The nearer streams, the Ramapo and the Mahwah, also the more distant Neversink, the Basher's Kill, the Mongaup, and the various other branches of the Delaware, all present the same questions of interstate rights that are found in the diversion of the Ten Mile river.

TABLE No. 74.

Possible Sources Incompletely Investigated, West of the Hudson in Southern New York.

From the map the following appear to be possible, but from brief reconnaissance most of them are found doubtful in quality of reservoir sites or impracticable, because of cost of development for reasons stated in following pages:

A closer survey, consisting of an accurate line of levels, with stadia surveys of basin areas and dam-site sections, with also a geological survey, and borings; would be worth its cost as a basis for closer estimates in case negotiations for the Housatonic water fail.

Watershed or River.	Point of Diversion.	Total drainage area of lowest promising point of diversion. Square miles.	First rough approximation at safe yield in million gallons per day with maximum storage practicable.	Approximate elevaiton above sea at this point. Present water level. About.	Roughly approximate distance by shortest practicable route from city limits near Jerome Park.	Direction of Flow.
Ramapo River	Ramapo	84.27	45	320	24	South into New Jersey.
Mahwah	Suffern	19.8	15	300	21	"
Popolopen Creek	Fort Montgomery	28	29	510	42	East into Hudson River.
Big Moodna	Salisbury Mills	123	95	320	49	"
Little Moodna	Woodbury Falls	11	7	420	49	"
Wallkill	New Hampton or Phillipsburg	417	250±	400	63	North into Rondout Creek.
Shawangunk	Bloomingsburg	47	30	440	71	North into Wallkill.
Rondout Creek	Ellenville	184	25	300?	75	North into Hudson River.
Bashers Kill	Port Orange	....	....	500	....	South into Delaware River.
Neversink	Quarryville (the Kitchen)	200	100	850±	60	"
Delaware (and Tributaries)	Above Port Jervis	3,600	....	420	60	South into New Jersey.

The watershed of Quassaic creek (29 square miles) was partially investigated but work discontinued on learning that steps had been taken for its development for the supply of Newburgh.

Safe Yield of the Ramapo River.

This watershed having been often advocated in years past as available for a water supply to New York will be first discussed.

It appears at present that the foremost difficulty about obtaining this water is that New Jersey cities are liable to need it in future for domestic supply, and that all the questions of the right to divert an interstate river would probably be raised in their strongest form and with very much less chance of being overcome than in the case of the Ten Mile river, where the question is mainly one of compensation for power.

There has been some tendency to overestimate the safe yield of the Ramapo river, both separately and in combination with the Mahwah. In the estimates for Brooklyn made in 1896, to obtain the 100 million gallons per day then sought, there were added to the Ramapo and Mahwah the following outside areas (taking reservoir locations as shown on Plate 2):

(See Report on Future Extension of the Brooklyn Water Supply transmitted to the Common Council, January 31, 1896.)

Drainage area from head of the Moodna.....	1.12	square miles.
" " " " " " Wanaque.....	13.84	" "
" " " " " " Wallkill.....	25.43	" "
Total	40.39	

The reservoirs, dams and points of diversion of these Wallkill branches are entirely outside the State of New York.

It is to be feared that the right to divert the 13.8 square miles of drainage area noted above from the Wannaque would be particularly difficult to obtain because it is tributary to the Passaic and is one of the sources upon which the East Jersey Water Company possibly may have to rely for future growth.

The area, as shown by the reservoir locations on Plate No. 3 of the Brooklyn report, to be taken from the—

Ramapo river was, by the map, only.....	73.97*	square miles.
Mahwah " " " " " "	19.80	" "
Total of Ramapo and Mahwah.....	93.77	" "
Total found by remeasuring of all the watersheds areas combined in the Brooklyn estimate of 1896.....	134.16	" "
Somehow, probably through accidental error, the footing in the Brooklyn report (p. 46), shows as.....	150.11	" "
—an overestimate of about.....	16	" "

* In Table No.75, page 498, the area that might be taken from the Ramapo River is increased by a proposed dam above Ramapo to 84.27 square miles. This dam was not contemplated in the Brooklyn Report.

It was estimated in this Brooklyn reconnaissance of 1896 that the Ramapo river, with the neighboring small watersheds on the Mahwah, Wallkill, Wanaque and Moodna, as noted above, could be relied upon with safety for 100 million gallons per day, and that entire cost of structures and rights for gathering, storing and delivering this into Ridgewood Reservoir would amount to only $14,930,206 (see p. 47, Brooklyn report).

Taking from the Brooklyn estimate all items of cost from the source over to a point on the pipe line opposite to the Park Hill Reservoir now proposed, I find these items to aggregate about $11,600,000 (these items do not include any allowance for a reservoir like that now proposed at Park Hill).

Revised Estimates on Ramapo River Supply.

From a careful study of this Brooklyn Report of 1896 in connection with the maps of the U. S. Geological Survey, the maps of the New Jersey Geological Survey, and C. C. Vermeule's Report on the Water-power of New Jersey, I am led to believe that the drainage areas, reservoirs and canals as proposed in this Brooklyn report could not safely be relied upon in a series of years of low rainfall to deliver over 73 million gallons per day, instead of the 100 millions then supposed safe, and that to construct the works delivering this 73 million gallons at the city limits, would cost considerably more than then estimated, or probably not less than $15,000,000, instead of $11,600,000 for the sources and the aqueduct as far as Park Hill.

This smaller yield and larger cost rest mainly upon the following matters of fact, which are open to any one who will carefully study the Brooklyn Report and the maps:

The said report forms such a prominent public record and has so often been quoted that an explanation of the difference in the two estimates appears proper.

1st. The contour maps of the U. S. Geological Survey and those of the Geological Survey of New Jersey, and the table in page 171 of Mr. C. C. Vermeule's Report on the Water-power of New Jersey; also Map No. 3 of the Brooklyn Report, all concur in showing the aggregate drainage area and some of the tributary areas to be somewhat smaller than quoted in said report, and apparently the area tributary to some of the small storage ponds is twice counted in said report.

2d. A large proportion of the main river is so poorly provided with storage capacity *at its lower end*, that in floods much of the water would escape over the Augusta Dam, and a large portion of the storage near the head waters has so small a tributary watershed that it will refill very slowly when emptied. No means is apparently provided by which a flood discharge on the main river could be conveyed into the large Mahwah Reservoir.

3d. While the estimates for cost of storage, $4,000,000, page 47 of the Brooklyn Report, may be intended to cover the following items, the text would imply that they had not been included. It will be noted that the capacity of all four artificial reservoirs, Mahwah, Augusta, Stony Brook and Round Pond, is estimated at a total of about 20.574 million gallons, and that on page 16 it is stated that cost of storage was assumed at $200 per million gallons. 20,000 mil. gal. @ $200 = $4.000,000. This apparently makes no account of the eight natural ponds of 3,890 million gallons and three artificial reservoirs of available capacity of 3,980 million gallons, that are to be flowed and drawn from 5 to 40 feet in depth, and apparently makes no account of the four reservoirs proposed on the headwaters of the Wallkill, of a total capacity of 2,345 million gallons (on page 167—Vermeule—it is stated that the water-power of this stream was once very fully developed for iron furnaces, but very little is at present in use).

It is not clear that the cost of the 14-mile pipe for conveying the 27 million gallons from these Wallkill tributaries into Little Long Pond is included, or that the very remarkable system of side-hill canals, 17 miles in total length, shown on Map No. 3, or the 1-mile tunnel from Ramapo to Mahwah Reservoir, are duly allowed for. It is open to serious question if this figure of $200 per million gallons allowed in the report for storage cost will cover all this, in addition to the numerous small reservoirs, or even cover the cost of the four main reservoirs alone.

It must also be remembered that steel plate for the long pipe line is very much higher now than when this Brooklyn estimate was made.

4th. Under the hydraulic grades as given in the Brooklyn report (about 2.80 feet per mile), the two steel pipes, 5 feet 2 inches in diameter, if, after a few years' use they were found to have the same coefficient of flow as the East Jersey pipe above Pompton, would convey only about 70 or 75 million gallons per 24 hours, instead of the 100 million gallons that would be delivered by a new, clean pipe.

Comparative Rainfall and Stream Flow in Ramapo District.

I am unable to find any long-term rainfall records especially applicable to the Ramapo Watershed. The best data bearing on this question appears to be that in Mr. C. C. Vermeule's admirable report on the Water-power of New Jersey. In the Brooklyn Report, page 41, is a general statement to the effect that the annual rainfall on the Ramapo is "given as 44 inches," which is the same figure referred to in connection with the Ten Mile Watershed, and on page 42 of the Brooklyn Report is the further statement that "The Ramapo Watershed is rocky, with steep slopes, which quickly carry off the rainfall." This last statement is well borne out by a study of the contour map.

While with this kind of a watershed the per cent. of run-off to rainfall may be high, as, for example, found by Mr. Herschel for a few years on the Pequannock (see Trans. Am. Soc. C. E., 1892, vol. 2, p. 288), this very steepness and imperviousness, or lack of sponge-like storage quality in the ground, of itself increases the requirement for a large volume of artificial storage reservoir capacity, if this water is to be held back and delivered at the slow, uniform rate required for domestic water supply. Conversely, if this ample storage volume is not provided, the floods will largely waste over the spillways of the dams and the safe yield of the system in time of drought be small, notwithstanding its high per cent. of runoff and an abundant total annual rainfall.

Principally for this reason, after having compared the gaugings of flow of the Ramapo river and its rainfall, given in Mr. Vermeule's report, with the rainfall and yield of the Croton and Sudbury for the same period, and having regard also to the rain gauges at Lake Hopatcong, as representing the precipitation in this highland region, I am led to believe that for the present purpose of reconnaissance the Sudbury Diagram, Figure 49 or the Croton Diagram, Figure 48, of the present report, gives as favorable a view of yield corresponding to a given storage volume per square mile as it is safe to entertain for the Ramapo region when estimating the safe yield in the greatest drought of half a century for purposes of domestic water supply.

It is but fair to state that the figures of monthly flow for a dry year given by Mr. Vermeule, page 129, for a New Jersey highland stream, give a larger *annual* run-off than that of the Sudbury in 1880, while giving substantially the same rate of flow for the five dryest months. Mr. Vermeule's estimate of storage requirement to produce a uniform flow of 666,000 gallons per square mile daily is 121.6 million gallons storage per square mile. The Sudbury requirement for 666,000 gallons daily per square mile is 130 million gallons storage per square mile, or not materially different, and by the Croton diagram, Fig. 48, the requirement is about 120 million gallons of storage or practically just the same as Mr. Vermenle's estimate.

Taking the drainage areas from the map and storage volumes as they stand in Table No. 2 of the Brooklyn report and computing the yield of each subdivision in proper relation to the storage upon it, we find that the entire Ramapo and Mahwah drainage areas of 93.77 square miles would under the same rainfall and ground flow as that of Sudbury in 1880–1883, yield only about 52 million gallons per day, which is only 554,000 gallons per square mile, instead of the 705,000 assumed as safe in the Brooklyn estimates, or only 79 per cent. as much. The report made to me by an assistant engineer, upon the topographical conditions above Ramapo Village, indi-

cates that by a greater expenditure, it is possible to recover a part of this discrepancy and to divert 10.3 square miles more of drainage, between Augusta and Ramapo, which apparently was not utilized in the Brooklyn project, but, in view of other pressing matters and the small probability that this source would become available, the field investigation was not carried farther.

Storage of Ramapo Flood Water.

Changing the plan of development slightly, so as to put the Ramapo project on the best possible footing by conveying the surplus flood water of the main stream into the great Mahwah reservoir for storage:

If a dam, say, 100 feet high, half a mile above Ramapo Village, could be allowed in addition to or instead of the proposed Augusta Reservoir, it would be practicable to construct such a flood water canal and tunnel of, say, 600 million gallons daily capacity, if given a slope of 12 feet per mile and made of 13.2 feet diameter, for about $400,000 per mile, lined and arched, or the two miles of flood water tunnel needed from Ramapo to Mahwah could be built for about $800,000.

As showing how large this tunnel would need to be to convey the flood waters into storage, we note on the Croton Watershed that a depth of 5 feet over the Old Croton Dam is so extremely rare that anything above could be allowed to waste without material effect on the storage value, and to waste all over 4 feet would not cause very serious loss. The Ramapo is probably a more "flashy" watershed than the Croton.

From a study of the Croton flood records and in proportion to the respective watersheds, a capacity of 600 million gallons per 24 hours appears an appropriate size for a flood water channel from the Ramapo river into the proposed Mahwah Reservoir, considering that the small reservoir at Augusta, holding 2,873 million gallons, and also the proposed diversion dam above Ramapo Village, would each aid as an equalizer. The tunnel could with safety be given the above mentioned declivity of 12 feet per mile, considering its intermittent use at full capacity, for the velocity in it, due to this great declivity, would be less than the velocity in the tail race tunnel at the Niagara water-power.

Only by allowing for the cost of such a tunnel and a diversion dam near Ramapo, does it appear justifiable to reckon the safe delivery of the Ramapo river itself at the high figure of 700,000 to 750,000 gallons per day per square mile; but, with this expensive addition of large tunnel and high diversion dam, we might be justified in reckoning the safe yield from the 104 square miles of the Ramapo and Mahwah at 65 to 70 million gallons per day; and if (as appears not at all probable), the water from the little Wanaque tributary and the Wawayanda and Long House Creek could be taken in, the whole yield might perhaps, by means of these additional structures and the larger area, be brought up somewhere near to the 100 million gallons per day at first estimated.

The Ramapo and Mahwah are much too small to furnish the supply now needed for Greater New York and would provide for hardly more than five years' growth of Manhattan and The Bronx, without regard to Brooklyn.

In the following computation it is *assumed* that the dam sites are safe and impervious; this being a matter not investigated by me.

SAFE YIELD OF RAMAPO WATERSHED AND ADJACENT LAKES AND RESERVOIRS.

TABLE No. 75.

Extended from Table No. 2 of Brooklyn Report on Additional Water Supply, 1896.

(See also Page 171—Vermeule—New Jersey Water Supply.)

Because of the unsymmetrical distribution of the storage, the yield of each tributary watershed must be computed separately and in connection with its own storage.

(The reservoir now suggested as perhaps possible just above Ramapo Village was not included in the Brooklyn Report. Figures are therefore given with and without this.) STORAGE RESERVOIRS PROPOSED IN BROOKLYN REPORT.	Drainage area as given in Brooklyn Report. Sq. miles.	Drainage area now found by tracing out and planimetering Geological Maps. — Where two or more reservoirs exist on same stream, the figures below exclude up-stream areas previously included. Sq. miles.	Water area as given in Brooklyn Report. Sq. miles.	Net land area per later measurements from Geological Map. Sq. miles.	Per cent. of Water Surface to Net Land Surface. — Per later measurements. Per cent.	Elevation of top of proposed dam. — Feet above mean tide.	Reservoir to be drawn down. Feet.	Capacity in million U. S. gallons.	Storage per sq. mile of net land area taken anew from maps. Million gallons.	Safe yield per sq. mile on Sudbury Basis limiting draft to 900 mil. gals. (because of injury to water if kept low too long). Thousand gallons.	Total safe yield this watershed with this storage on Sudbury basis. Diag. 49. Million gallons per day.
1	2	3	4	5	6	7	8	9	10	11	12
Round Pond, mainly artificial	.70	.70	.400	.30	133 per cent.	707	55	2,656	8,850	900,000	0.3
Mt. Bashan Lake, natural	2.92	2.91	.450	2.46	18.3 "	851	14	756	307	855 000	2.1
Slaughter Pond, natural	.62	.70	.170	.53	32.1 "	1,050	5	140	264	740,000	0.4
Cranberry Pond, natural	.46	.42	.100	.32	31.3 "	1,013	5	100	312	820,000	0.3
Island Pond, natural	1.06	1.30	.110	1.19	9.2 "	908	10	210	176	710,000	0.8
Augusta Reservoir, artificial	53.15	47.10	.620	46.48	1.3 "	480	40	2,873	62	430,000	20.0

Long Pond, natural	1.14	.93	.160	.77	20.8 "	830	15	661	918	900,000	0.7
Stony Brook Reservoir, artificial	18.64	17.07	.400	16.67	2.4 "	465	30	2,210	133	660,000	11.0
Negro Pond (Portage), natural	.80	.81	.140	.67	20.9 "	620	20	441	658	900,000	0.6
Portage Reservoir, artificial	3.73	2.03	.130	1.90	6.8 per cent.	540	40	526	277	850,000	1.0
Additional on Ramapo above Ramapo	....	10.30	(1.75)	8.55	20.5 "	420 (?)	25 (?)	(7,000)	820	900,000	7.7
Mahwah	24.25	19.80	3.654	16.15	22.6 "	400	20	12,835	795	900,000	14.5
Total proposed per above list on Ramapo and Mahwah	.....	104.07	8.08	95.99	8.4 per cent.	....	....	30,408	317	.....	59.4
Total exclusive of Ramapo Reservoir	107.47	93.77	6.33	87.44	7.3 "	....	....	23,408	268	.....	51.7
Little Long Pond (Moodna Creek)	1.24	1.12	0.330	0.79	41.7 per cent.	707	20	882	1,120	900,000	0.7
Wawayanda Lake*	8.00	6.35*	.820	24.61	3.3 per cent.	(?)	(?)	2,345	95	570,000	14.
Wawayanda Reservoir	7.00	6.86									
Long House, Upper Reservoir	8.70	6.56									
Long House, Lower Reservoir	4.30	5.66									
Total from Wallkill Tributaries	28.00	25.43	.820	24.61	.....	....	....	2,345	95	.....	14.
Sterling Lake	4.60	4.95	.450	4.50	10 per cent.	749	10	700	155	665,000	3.0
Sterling Reservoir	6.50	6.68	(.110)	6.57	1.7 "	600	15	(310)	47	375,000	2.5
Eagle Valley Reservoir	2.30	2.21	.330	1.88	17.5 "	550	10	264	140	615,000	1.8
Total from Wanaque Tributaries	13.40	13.84	.89	12.95	6.9 per cent.	....	....	1,274	98	.....	6.7
Total from watersheds per above list, including Ramapo Reservoir	.....	144.46	10.12	134.34	7.6 per cent.	....	....	34,909	260	.....	80.8
Total from watersheds proposed in Brooklyn Report	150.11	134.16	8.37	125.79	6.7 "	....	....	27,909	222	.....	73.1
The over wash from one reservoir into another lying below it on same stream, may in the few cases where the upper reservoir is small, add enough to bring the whole safe yield with watersheds and reservoirs of Brooklyn Report to perhaps											75.0

* The areas given above for the Wawayanda and Long House watersheds are based on the reservoir locations on Long House creek indicated on Plate No. 2 of Brooklyn Report; if these watersheds are measured as shown on Plate No. 1 of same report, then the areas will aggregate 12.5 square miles less than given above, and the yield will be correspondingly less. The reservoir locations shown on these plates do not coincide, but the plate showing the larger area has been assumed here to be probably the one on which estimates were based.

Other Watersheds West of the Hudson at Suitable Elevation to Deliver a Gravity Supply to New York at 300 Feet Head Above Sea Level at City Limits.

The Ramapo river and the small watersheds adjoining, have been already considered by themselves, because of their having been separately discussed in previous reports, and indeed it still appears that if these were to be brought to New York they could best be conveyed in a conduit built for them alone, and apart from other supplies.

Taking now a broad view of the whole territory west of the Hudson and within reach of New York to see what hopeful possibilities in water sources it may contain, we are at the outset embarrassed by the absence of any good topographical maps.* We can, however, make substantial and rapid progress in our inquiry in seeking a source of water worthy of consideration for the supply of New York by attention to a few general principles.

First—We must seek natural watercourses draining large areas. The only original, perennial source is the rainfall; the rainfall on any hundred square miles in New York State is pretty much the same, and the total annual run-off, anywhere in the region to be considered, will probably be not very far from half of the total annual rainfall.

However fortunate in storage reservoirs a stream may be, a safe yield so large as a million gallons per day per square mile in extreme drought is practically unknown, and if we need ultimately 500 million gallons per day, we must find watersheds aggregating from 700 to 2,000 or more square miles according to the reservoir sites.

Second—Opportunity for the economical construction of enormous storage reservoirs is hardly less important than great drainage area. Without storage reservoirs the safe yield of the Croton would be less than 50 million gallons per day, while, with its 200 million gallons of storage per square mile, its safe yield will be more than sixfold the natural low-water yield, perhaps tenfold.

Third—It is very desirable that the elevation of the stream at the point of diversion, either now or after the dam is built, should be such that the water can flow to New York by gravity without pumping, and be delivered at the city limits at an elevation of 300 feet above the sea.

From the existing maps the watershed areas can be measured with all the precision needed for these preliminary studies.

The information not given by the present maps, but which would be given by the regular topographical maps now slowly in progress, and which we so much need, concerns the broad valleys that could be dammed up to

*The excellent topographical maps made under the joint authority of the New York State Engineer and the United States Geological Survey have only touched the edges of this region. To gain information about the elevation of the principal valleys, the Chief Engineers' offices of several of the railroads were visited, and profiles were very courteously furnished us. The admirable geological surveys of the State of New Jersey were of much aid along near the southern border of New York. The Chief Geographer of the United States Geological Survey very kindly furnished us with an advance proof of the Schunemunk Quadrangle covering a considerable part of the Moodna Watershed. The topographical maps already issued, although they give but a narrow strip along the Hudson, were also found of much use. As elsewhere stated, it would pay for New York City to add to the State's appropriation if need be, to rush the completion of these topographical maps for all regions in which water may be sought.

form storage reservoirs, and the elevations of the valleys above sea level. We therefore sought this information by such inspection and reconnaissance with aneroid barometer as the time afforded.

When making the preliminary study from maps, after the possible watershed has been found and its possible reservoir sites picked out, then there remains the question of the practicability of dams, the tightness of substrata and the questions of swamp water, algæ and pollution.

The data already available gives enough positive information so that we can quickly cut out certain watersheds as hopeless and can mark others as hopeful and worthy of examination in the field. The outlook from the office study of the maps is commonly more hopeful than the results of the field examination when questions of safety and difficulty of dam sites, volume of storage, extent of shallow flowage and quality of water come more prominently into view.

From the very fact of incompleteness of our studies in this region there will thus be a natural tendency to take too hopeful a view of the possibilities unless this is carefully guarded against, and even then it may come in. *Therefore, we may feel doubly well assured that having found by our study of maps and our incomplete reconnaissance that every one of the possible watersheds west of the Hudson is less favorable than the Housatonic project, without further delay and without fear of mistake we may at once make earnest effort to secure a clear title to the Housatonic source,* meanwhile continuing investigation of the sources west of the Hudson only for the purpose of saving time, in case the legislative difficulties in the way of the Housatonic should not be promptly solved or the price asked be found too high.

Turning now to Map No. 1 (frontispiece), or preferably to a general map on a larger scale, and studying the possibilities in the light of the principles mentioned above, we note first that the elevation of many broad, abundant watersheds is entirely suitable for the delivery of their water to New York City at elevation 300 by gravity flow.

The slope of the New Croton Aqueduct, 0.70 feet per mile, may be regarded as about the smallest that economy will permit—2.0 feet per mile, if obtainable, is much more economical in aqueduct cost, as will be noted by a study of Diagram No. 66.

From the map and the railroad profiles it will be found that nearly all of the large drainage systems in southern New York, west of the Hudson, can be tapped at points of suitable elevation to allow for an aqueduct slope or declivity of more than one foot per mile and have the delivery end at elevation 300.

Beginning at the New Jersey line and proceeding west and then north we note:

First—The Mahwah river at Suffern, and the Ramapo river at Ramapo, are each at about elevation 320, and but little more than 20 miles from the city limits, and with the raising by dams would readily afford the required elevation.

Second—The Wallkill, the next important stream met in going westward, has an elevation not far from 400 feet above sea, where it flows northward across the New Jersey line, and continues a swampy, sluggish stream with but little less elevation down as far Phillipsburgh. So that so far as quantity is concerned, it, with its tributaries, Longhouse, Warwick, Pochuck and Rutgers creeks, in all 415 square miles, can be led by gravity from a point of diversion 400 feet above sea level, distant from the city limits about 63 miles, along a conduit line and therefore affording a hydraulic gradient averaging about 1.50 feet per mile, which is ample, to a terminal elevation of 300 feet above sea.

Third—Next, the main Delaware river, draining 3,600 square miles, is reached at Port Jervis, at an elevation of about 420 feet above sea—60 miles from New York—and so far as elevation is concerned favorable for gravity flow to New York by a 60-mile tunnel (or only twice as long as the New Croton tunnel), and having a declivity of about 2.0 feet per mile.

This elevation of the Delaware is a very important consideration as proving that all of the large territory which it drains, comprising everything west of the Wallkill and Rondout until we get beyond economic reach of aqueducts, is at an elevation suitable to deliver high-pressure water to New York.

Fourth—Northward the Esopus has already been considered in Appendix No. 9, with the Catskill and Schoharie creeks, which, draining the northern side of the Catskill mountains, drain a large territory of suitable elevation.

It is, therefore, plain and certain that a requirement to deliver water at 300 feet elevation above sea level, will work no great hardship in seeking an ample source, and is plain that ample drainage areas above this elevation can be found, and that the question of a supply from west of the Hudson will quickly resolve itself into questions of good reservoir locations, practicable dam sites, quality of water and comparative cost of development. The study, therefore, naturally next proceeds with a consideration of the possibilities for storage on those streams lying nearest and thus admitting of the shortest and cheapest aqueduct lines and also first considering those that are free from all questions of the right to divert an interstate river.

The Popolopen Source.

Going northward from the Ramapo and Mahwah, already considered, the first possibility offered by the map of a drainage area large enough to be worthy of consideration for the present problem* is Popolopen creek,

* The small drainage areas above the required elevation of about 7 square miles on Cedar creek, near Haverstraw, should, apparently, be reserved for the future needs of Haverstraw. Minnescecongo creek is already, in part, appropriated for the water supply of Haverstraw, and for compensation-water for the Garnerville Print Works. The small head waters of the Hackensack also can hardly be considered as available for New York City, for if their diversion is allowable, it would be more appropriate for the small towns on the west margin of the Hudson, like Nyack, which is understood to have already taken water from this source.

which falls into the Hudson near Fort Montgomery. This was one of the sources promoted by the Ramapo Company and specifically mentioned by Water Commissioner Dalton in discussing additional supplies (see "City Record," Wednesday, August 23, 1899, p. 5271), and so far as I have had opportunity to carry the present investigation, it looks like a very favorable watershed of its size, but too small except as an adjunct to some larger source. Of itself it will yield, safely, hardly more than 20 million gallons per day, or but little more than enough to take care of one year's increase in New York's water consumption.

The report of Mr. Ropes upon this watershed, slightly abbreviated and amended, is as follows:

Popolopen Watershed.

This watershed is situated in the heart of the Highland District, surrounded by rugged hills, ranging from 1,000 to 1,500 feet high. It has an available area of only about 28 square miles, and the streams draining it are, therefore, comparatively small ones. The geological formation is principally gneissic rocks, and the conditions of topography, geology and settlement are all remarkably favorable to the purity of its waters. Popolopen creek enters the Hudson river at Fort Montgomery, and its point of diversion would be about 30 miles, in a direct line, from city limits. For the first two miles from the river, the valley rises very rapidly, reaching an elevation of 500 feet in that distance. Above this point the declivity of the valleys is generally much less, and in places they open out into small basins of from ¼ to ½ mile in area. These basins are quite a prominent feature of this watershed, and I should say that as many as 10 or 12 suitable sites could be found for reservoirs of the above-mentioned area. There are 5 natural ponds in this district, which, taken together, may aggregate ¼ square mile of water surface. Popolopen and Mine ponds are largely artificial, having been improved by the Forest-of-Dean Mining Company, to obtain water-power for hoisting and pumping in their mining operations. No mining has been done at this place for four years or more, but a caretaker remains in charge of the plant, so it does not appear that the company has wholly abandoned the mines.

I explored only the north half of this watershed, going to Round, Long and Popolopen ponds, and following down the main stream to Fort Montgomery.

I understand that Round Pond, with its drainage area of about 0.13 square mile, has been appropriated as a water supply for the Military Academy at West Point. This pond was drawn very low during the drought of last summer and fall. It had not entirely refilled during the winter, and at the time of my visit was still 5 or 6 feet below high-water mark.

Long Pond has an area of say 30 acres, with a watershed of 1.4 square miles. Its present flow line can easily be raised 20 feet by a dam 30 feet high (above ground) and with a length of 300 feet at bottom and 800 feet at crest, giving a water area of about 80 acres, and an available capacity of 320 million gallons. Ledge is close to the surface in the side bluffs; there was no evidence of ledge in the swamp across the bottom, but I am inclined to think it shallow, say 10 or 15 feet.

Popolopen Pond has an area of about 150 acres. The existing dam is about 15 feet at highest part. There is a good site just below this for a dam 50 feet high, by length of 300 feet on bottom and 900 feet on crest, which would raise the pond level 30 feet. Ledge is exposed entirely across the bottom at this site. It is a tight, massive rock, with the strike parallel to the face of the dam and therefore particularly favorable for foundation. The watershed above dam is 5.7 square miles. Raising the flow line of this pond 30 feet would give a probable area of 250 acres, or 0.4 square mile, with an available storage of nearly 1,900 million gallons.

About ½ mile below Popolopen dam, at the stone arch bridge, where the road crosses the brook, I observed another favorable location for a dam about 40 feet high, with bottom length of 150 feet, and crest 700 feet long. Ledge is exposed clear across bottom. The character of the ledge and the direction of its strike are the same as at the Popolopen dam. This would flood a swale of say 150 acres. There is a gap in the hills on the south side which might require a short earth dyke to complete the inclosure. The tributary watershed is 3.2 square miles; available capacity of reservoir would be about 600 million gallons.

At Mine Pond there is very little chance for additional storage by raising the present dam.

Proceeding down stream, there was apparantly a good opportunity for a reservoir with a dam 50 or 60 feet high, and say 1,000 feet long on crest, at a point about ½ mile below Wyant's farm house. This site was nearly ½ mile from the road, and, as it was growing late, I did not go on to the ground; but I could get a good, open view of it, and should say that general indications were good for finding ledge at moderate depth. This dam if found practicable of height stated above, would flow back nearly to the Forest-of-Dean Works, making a reservoir averaging ¼ mile wide for two miles in length. Its available storage would be about 1,700 million gallons for a watershed of 13.9 square miles. From what I saw of this watershed my impression is that on each of the larger branches of this creek two or more places can be found for dam sites of from 30 to 50 feet high, which will flood from ⅛ to ¼ square mile of area. It also appears not improbable that the other ponds could be raised to the same extent as has been suggested for Long Pond.

Mr. C. C. Vermeule, in Volume 3 of the Geological Survey of New Jersey, page 129, estimates that the total annual run-off from the watersheds of the New Jersey Highlands in the driest year would be equal to 16.82 inches, or at the rate of 811,000 gallons daily per square mile.

For the Croton watershed it has been concluded (see page 23) that with a storage of about 200 million gallons per square mile, a safe yield of 275 million gallons per day, which is 763,000 gallons per square mile of gross area, is safe, and that it will not pay to develop storage beyond that limit. Accepting the same limits for the Popolopen we have a storage requirement of 5,300 million gallons, and a safe yield of 21 million gallons per day.

The following table shows the storage requirements of the different divisions of the Popolopen watershed, to furnish a sure yield of 800,000 gallons per square mile, and to what extent these requirements are met by the several reservoir sites reported on above.

* Name of Reservoir.	Area of Watershed.			Per Diagram No. 49.	Probable Available Storage.	Excess or Deficiency of Storage that this Reconnaissance Shows Available.
	Net Land. Square Miles.	Water Surfaces. Square Miles.	Total Square Miles.	Required Storage to Maintain Safe Daily Yield of 800,000 gallons per Day.	Capacity of Proposed Reservoir.	
Long Pond	1.3	0.1 (8%)	1.4	(224) 291 m.g.	320 m.g.	− 29 m.g.
Upper Popolopen	5.2	0.5 (10%)	5.7	(229) 1,191 "	1,860 "	+ 669 "
Lower Popolopen	2.9	0.3 (10%)	3.2	(229) 654 "	640 "	− 24 "
Fort Montgomery	13.4	0.5 (4%)	13.9	(214) 2,863 "	1,740 "	— 1,128 "
Rocky Swamps	3.9	0.2 (5%)	4.1	(217) 846 "	1,080 "	+ 134 "
	26.7	1.6 (6%)	28.3	5,860 m.g.		

* The locations of the above-proposed reservoirs are shown on the Schunemunk sheet of the State topographical map, surveyed last summer.

From the foregoing table it will be seen that the principal deficiency of storage provided is in the watershed of the Fort Montgomery Reservoir. From a study of the topographic map, together with my general knowledge of this section, I feel quite confident that two or three additional and equally favorable reservoir sites can be found in Deep Hollow and Queensboro, valleys, which will furnish the desired storage and bring the safe yield of the whole watershed up to the same figure as the Croton (763,000 gallons daily per square mile).

GENERAL CONCLUSIONS.

The valleys of the Popolopen watershed are narrow mountain valleys, occasionally opening out into swales or basins generally of ¼ to ½ square mile in area, and generally at such points favorable sites can be found for dams from 30 to 50 feet in height. At the places observed, indications were generally good for rock foundations for the dams. It appears that enough such reservoir sites may be found to furnish a certain supply of 20 million gallons daily from the 28 square miles. The conditions affecting the quality of the water appear most favorable, and this promises to be a supply of exceptional purity. The principal reservoir sites would be Fort Montgomery (area, 0.45 square miles; available capacity, 1,700 million gallons) and Popolopen Pond (area, 0.40 square mile; capacity, 1,900 million gallons).

COST OF STORAGE WORKS ON POPOLOPEN CREEK.

Proceeding then on the assumption of 20 million gallons daily supply from this source, I present the following *very rough* estimates of the probable cost of the dams and reservoirs needed:

Fort Montgomery Reservoir.

Say dam 80 feet high by 1,000 feet long on crest, probably not more than 300 feet long in the bottom. Rock generally at shallow depth. Masonry overflow 150 feet long and 70 feet high. Earth embankment with core wall—cost complete	$600,000 00	
380 acres of land, at $80	30,000 00	
100 acres of grubbing and clearing, at $50	5,000 00	
2 miles of new roads, at $10,000	20,000 00	
6 buildings, at $2,500	15,000 00	
Dam and canal to divert waters of Queensboro Brook into this reservoir.	75,000 00	
		$745,000 00

Lower Popolopen Reservoir.

Dam 40 feet high by length of 700 feet on crest and 150 feet in bottom. Ledge exposed in bottom and sides. Earth dam, with core wall, with overflow 50 feet long over ledge at side	$140,000 00	
200 acres of land, at $80	16,000 00	
30 acres of grubbing and clearing, at $50	1,500 00	
1 group of buildings, $3,500	3,500 00	
New road (none)	.....	
Dam and canal to divert Twin Lakes Brook into reservoir	25,000 00	
Possible contingency of dikes at one or two gaps	50,000 00	
		236,000 00

Upper Popolopen Reservoir.

Dam, 60 feet high by 300 feet long on bottom and 900 feet long on crest. Ledge exposed clear across. Overflow at side 30 feet long. Earth and loose rock dam, with core wall	$220,000 00	
350 acres of land, at $40	14,000 00	
150 acres of grubbing and clearing, at $50	7,500 00	
Buildings	None.	
New highway	None.	
Damage to Forest-of-Dean Mining Company (perhaps)	50,000 00	
		$292,000 00

Long Pond Reservoir.

Dam 30 feet high, 300 feet long in bottom and 800 feet long in crest, say 20 feet to ledge across bottom. Earth dam with core wall. Overflow 30 feet long	$105,000 00	
100 acres of land, at $60	6,000 00	
20 acres of grubbing and clearing, at $50	1,000 00	
1 group of buildings, $3,000	3,000 00	
1 mile new road	10,000 00	
		125,000 00
Rocky Swamp Reservoir, and such others as may be built in Deep Hollow and Queensboro valleys, to bring the yield up to 800,000 gallons per square mile, may be assumed to cost the same, per million gallons of storage, as the above, or 2,000 million gallons storage, at $310 per million gallons		620,000 00
		$2,018,000 00
Add 20 per cent. for contingencies and supervision		404,000 00
		$2,422,000 00

The quantity of water to be derived from this source is probably not sufficient to warrant its being carried to city limits by an independent conduit, and the matter of cost for transportation will be taken up later in connection with that of the adjacent watersheds.

QUASSAIC WATERSHED.

This is another small watershed lying at suitable elevation for gravity flow of about the same area as the Popolopen, falling into the Hudson river about 15 miles farther north. If available the probable point for diversion would appear to be about 3 miles northwesterly from Newburgh.

The area above this point is about 29 square miles. A line of barometric levels was carried across this district, which showed the possibility of diverting the greater part of the flow of this stream, at some point not farther than 2 miles above the Powder Works, and flowing it by gravity to New York. Later, it was learned that the City of Newburgh had taken steps towards the development of this source of supply for its own use, so further investigation in that direction was dropped, and this area is not considered as a possible source.

MOODNA WATERSHED.

This is a large creek which empties into Hudson near Cornwall. The water level of the main stream, just above Salisbury Mills, and about 49 miles from the New York City limits by the shortest economical conduit line, is practically at 300 feet elevation. Obviously, this is at least 50 feet too low for selection as the point of diversion for these waters by gravity flow to 300 feet elevation in New York; but, since there is a very favorable site there for a dam at least 50 feet high, providing the drift at the sides is shown by test-pits to be safely impervious, and which might be made 80 feet without exceeding a reasonable cost; if the ledge at the site of the wing walls is within reach or the drift impervious, it appears possible with the higher dam, or with the lower dam supplemented by low-lift pumps, to utilize the whole watershed to this point; and in order to make as good a showing as possible, it has been so treated. With the foregoing assumption, the area of the Moodna watershed available would be:

On Big Moodna Creek, above Salisbury Mills.....	123	square	miles.
On Little Moodna Creek, above Woodbury Falls.....	11	"	"
Total.....	134	"	"

Mr. Ropes' report of his reconnaissance, abbreviated and amended by some comments of my own, is as follows:

Salisbury Mills is, in a direct line, about 38 miles from city limits. The actual length of conduit would probably be 11 miles greater than that, as, with our present information, it is difficult to conceive a nearly direct route through the Highlands that would be economical, and it appears as if the conduit must make a detour around the broadest and highest portion of this group of hills.

I did not visit the upper portion of Little Moodna Creek, so have no personal knowledge of that section. The lower portions of the valley, where I observed them near Mountainville, are filled with heavy clayey gravel deposits. The water in the stream looked clear and pure. I should judge from the map that this area may be classed as a "Highland" watershed, though I gather from the "Fifteenth Annual Report of the State Geologist," that the geology of this section is very different from that of the Popolopen watershed. Thus on about ⅓ of it, shales, shaley sandstones and Schunnemunk conglomerate predominate, another ⅓ of the area is of limestone formation, and the remaining ⅓ is gneissic.

For the development of this Little Moodna watershed, the map indicates a very promising place for a dam at Woodbury's Falls. Slaty ledges outcrop at the base of the mill dam at that point (see "Report of New York State Geologist, Geology of Orange Co.," p. 417). The basin above this dam site is shown on the map as quite extensive. It is such a prominent feature of the topography that I think we may safely accept the evidence of the map. With flow line at 480, the Village of Central Valley would probably not be disturbed. According to the map this reservoir would have an area of about 0.55 square mile, and a capacity of 1,400 million gallons.

With 7 per cent. water surface, per Diagram No. 48, this storage of 1,400 ÷ 11 — 127 million gallons per square mile would maintain a yield of 650,000 gallons per square mile, or a safe daily supply of 7 million gallons. It does not appear reasonable to estimate upon a flow line at elevation 500. If the flow line is assumed at elevation 500, then Central Valley would be drowned out. While the area of the reservoir would then be 1.45 square miles, and its capacity5, 400 million gallons, or 490 million gallons per square mile, and its theoretic yield (with 16 per cent. of water surface) a daily supply of nearly 10 million gallons; it appears hardly prudent to estimate on so high a rate of draft. It appears probable that with the smaller reservoir, together with the improvement of Cromwell Lake, there could be had a safe yield say of 750,000 gallons per square mile of gross area, giving a daily supply of 8 million gallons from this Little Moodna creek watershed. I assume this latter quantity to be the maximum reasonable development. The sanitary conditions of this watershed were not investigated, nor the character of the subsoil of the reservoir site.

Big Moodna Creek.

Relative to developing the 123 square miles on Big Moodna creek, reference has already been made to the attractive location, just above Salisbury Mills, for a dam and large reservoir (see Figure 100 for a roughly approximate cross-section of the valley at the proposed dam site). While making my observations in the field, I had in mind a dam of not more than 50 or 60 feet in height, but with the better comprehension of the ground which I have since obtained by studying the proof of the topographical sheet of this section since received, I can see that if test pits show the bed rock to rise in the banks, or if it holds enough clay to make it surely impervious, then a possibility may exist for obtaining a remarkably large volume of storage by building a higher dam (say 80 feet high). The topographical map just mentioned was not received from Washington until my return from this trip, so it was not available for field use.

Working out this proposition for the 80 foot by the Sudbury Diagram, No. 49, and by the Croton experience, we have—

With flow line at 370, area of full reservoir.....12.35 square miles.
With reservoir drawn down to 350, area..... 3.9 "
Capacity of reservoir between 350 to 370.....32,480 million gallons,

or 254 million gallons of storage per square mile, which is somewhat more than the Croton will have in 1902. If the reservoir bed and the dam site prove impervious, we may safely assume the same yield per square mile as deduced for the Croton, whence this 123 square miles would yield $\frac{123.}{360} + 275 = 94$ million gallons per day, or the Big and Little Moodna together would yield $8 + 94 — 102$ or say 100 million gallons per day.

This suggestion of a single large reservoir for Big Moodna Creek at Salisbury Mills should be regarded as only one of several plans which might be adopted. Westward from Salisbury Mills, this watershed is a broad, rolling country, with wide, shallow valleys separated by low hills and ridges; and my casual examination led me to believe there were many basins where shallow reservoirs of considerable extent could be made. For example in the vicinity of Chester there appeared to be such a basin. From the map it appears that a 40 to 50 foot dam at Burnside would flood a large area.

I examined the valley of the main stream carefully from Orr's Mills to above Washingtonville. Westward from that point, the broad views I had obtained from the train, together with the very marked characteristics of the country, made it seem unnecessary to cover this ground with more detail. That the main valleys in many places, and for long stretches, have only a slight fall, was easily apparent; and this is a condition favorable to extensive flowage with low and inexpensive dams. The question of the tightness of these dams and the imperviousness of the substrata remains for a less hurried and more expensive research.

Possible Poor Quality of Moodna Water.

The geological report of Orange County has this to say of the ancient lake beds, which applies to some of the depressions of this district (see p. 474):

" Of importance among the quartenary deposits of Orange County are the numerous old lake beds, which especially abound in the Hudson river area. The lakes were formed by the damming up of the valleys and depressions between the slate ridges, and they disappeared through the subsequent filling up of their basins, or the cutting down of their outlets. Black soil underlies the surface to a depth of from 5 to 50 feet, and this, according to Mather, is, in turn, underlaid by marl.

Other important swamp areas are: Greycourt Meadows, covering 500 acres and extending from Craigsville to Chester, Black Meadow in Goshen Township, 1,000 acres; Tamarack and Purgatory Swamp in Hamptonburg."

The last-named meadows or " swamps " are all in this watershed.

The prevalence throughout this district of these swamps and meadows, with their deep covering of black muck, raises a doubt as to the desirability of water from this source. Relative to that question, I can only say that at the time of my visit, the water in the streams presented a good appearance. The rock formation which prevails over nine-tenths of this watershed is the Hudson river shales and slates.

Some serious objections apply to the large reservoir suggested above Salisbury Mills, such as the flooding out of Washingtonville, a thriving village of 600 or 700 inhabitants; also the large amount of shallow flowage which would be exposed when the reservoir was drawn down; the great area of fertile land submerged, and the extensive changes of railroad and highways. The shallow flowage could be regulated to a considerable extent by low dams across different arms of the basin.

The proper development of the Big Moodna watershed for purposes of public water supply is a problem which would demand much investigation and study for its solution, and the present assumption of a single large reservoir above Salisbury Mills is only for the purpose of nearing some conclusion as to the probable cost of a water supply from this source and whether or not it is out of the question.

If a dam 80 feet high at Salisbury Mills should be proved to be impracticable, then it would be necessary to sacrifice perhaps one-third of the watershed, or to resort to lifting the water 20 to 30 feet with pumps. For the

present, however, adhering to our assumption of an 80-foot dam, I present the following guess as to the probable cost of this dam and reservoir.

Salisbury Mills Dam and Reservoir—

Flow line 370

Drawn down to 350.

Dam 85 feet high, main portion 300 feet long in bottom and 1,100 feet long on top, with say 1,200 feet of earth dikes averaging 25 feet high. Also from the map, it appears that two dikes each about 1,000 feet long and not more than 20 feet high would be required at low gaps northward from the main dam site. Length of masonry overflow assumed as 400 feet. Slate and shale outcrops in bed of river at this site, but side bluffs are apparently drift to a very considerable depth. Imperviousness not known.

Probable cost of dam, overflow, gate-house and side dikes, complete, say	$850,000 00
Low dams for regulating shallow flowage, say 5,000 lineal feet, averaging 15 feet high, with 800 lineal feet masonry overflows, sluiceways, gates, etc., complete	250,000 00
Land damages—	
2,600 acres at $300	780,000 00
7,500 acres at $100	750,000 00
Grubbing and clearing 2,000 acres at $70	140,000 00
Buildings, say rather more than for Ten Mile Reservoir	1,200,000 00
Water power at Salisbury Mills and Orr's Mills, say 600 Horse Power, at $500	300,000 00
Relocation of railways, 24 miles at $18,000 per mile	432,000 00
Relocation of highways, 65 miles at $12,000 per mile	780,000 00
Embankments and bridges across reservoir for highways, 12,000 feet, say 30 feet high and rip rap	700,000 00
Possible contingencies, such as removal of cemeteries, legal expenses, damage to established business and extra compensation to railroads	750,000 00
	$6,932,000 00
Add 15% for supervision and contingencies	1,040,000 00
	$7,972,000 00

This is at the rate of $245 per million gallons for 32,480 million gallons storage, and this figure appears reasonable when we take into account the good quality of land submerged, the towns and buildings destroyed, and the many miles of railroads and highways to be rebuilt.

For necessary storage works on the Little Moodna allow 2,270 million gallons x $300 ..... $681,000 00

In the latter case, judging the situation solely from its appearance on the Schunemunk sheet of the State map, and giving due weight to the necessary changes of highways and railroad, as well as the partial flooding of the villages of Woodbury's Falls and Highland Mills, and possibly a part of Central Vailey; also sewerage of Central Valley, it seems improbable that the Little Moodna watershed can be developed for a daily supply of 8 million gallons at a *less* cost than $300 per million gallons for the storage works, and the cost would be more likely to prove 25 per cent. larger than that.

Conduits for Moodna and Popolopen Supplies.

With the foregoing assumptions, there appears to be a *possibility* of obtaining daily the following volumes of water from—

Big Moodna Creek, above Salisbury Mills.....	92	million gallons.
Little Moodna Creek, above Woodbury's Falls.....	8	" "
Popolopen Creek, above Fort Montgomery.....	20	" "
Total safe yield per day.....	120	" "

If the quality is all right this quantity would be of sufficient magnitude to warrant the construction of a conduit for its transportation to New York City.

This would require a conduit about 49 miles long from Salisbury Mills to city limits, with a branch 5½ miles long to Woodbury's Falls, and a branch 3½ miles long to Popolopen Creek.

On the topographical map (not reproduced) I have sketched what there appears to be the best general location for these conduits. The ground over which the greater part of this line runs has not been inspected in the field.

In laying these lines, the aim has been to get as much cut and cover aqueduct, and as little pipe line, as seemed practicable.

The route, as sketched on the map, keeps on the north side of the Big Moodna Valley to a point abont ¼ of a mile south of Cornwall Station, thence crosses Moodna creek by steel pipe siphon to the high ground on the east side, where, after about 1 mile of cut and cover aqueduct, it crosses through the north portion ot the Highlands by a tunnel 4 miles long under Black Rock Hill, and with the eastern portal at Highland Falls; thence it crosses the Hudson river with steel pipe siphon, probably in tunnel under the river, at the most suitable point between Highland Falls and Fort Montgomery. A crossing in the vicinity of Con's Hook Light appears the most favorable for general direction, also for a connection with the Popolopen Branch. From the east bank of the Hudson, the line proceeds in a southeasterly direction, with short tunnels under the hills, steel pipe siphons across the valleys, and short stretches of cut and cover aqueduct wherever the slopes will permit of that construction, thus passing about ½ mile southwesterly from the village of Putnam Valley, to the site of the proposed Holman's Mill reservoir, about 3½ miles east of Peekskill. From Holman's Mill reservoir the line proceeds in a southeasterly direction, crossing Croton Lake near the old Croton Dam; thence through the pass at Merritt's Corners, and then keeping on the high ground to the eastward of Pocantico, Saw Mill river and Grassy Sprain Valleys as far as possible at the required elevation for cut and cover aqueduct, to the proposed Park Hill Reservoir, and one mile farther to the city limits between Woodlawn and Lincoln. It seems fairly probable that about 17 miles of the 27.5 miles from city limits to Holman's Mill, could be built as cut and cover aqueduct.

The branch line from Popolopen creek leaves the Fort Montgomery reservoir by a short tunnel, and then follows the trend of the bluffs in a northeasterly direction to a junction with the main line at a point about 1 mile southerly from Highland Falls. The branch line from

Woodbury's Falls is located on the east side of the Little Moodna Valley at a suitable height for cut and cover aqueduct.

If it appears desirable to utilize all three supplies, then the tunnels and cut and cover portions of the main lines, below each junction, should be of sufficient capacity to carry the amount from the combined sources above such junction, so that one conduit may serve for all three supplies, whether they be taken all at the same time, or in part deferred. In order, however, to show clearly the cost of taking in the two smaller watersheds, the following estimate (see page 513) gives, in the first column, the cost of the main line conduit proportioned to carry only the supply from Big Moodna; and the parallel columns show the increased cost due to enlarging the conduit at the start to admit the supplies from the Popolopen and Little Moodna.

A very Rough Estimate of Cost of 120 Million Gallons per day from the Moodna and Popolopen Creeks. Estimated Cost of Conduits to City Limits.

Assumptions: Hyd. grade elevs. 350—303. 32.5 miles aq. and tunnel, at 0.5 pr. m. 16.3' 15 miles pipe line, at 1.7 pr. m..... 25.5' 1.5 miles Hudson river, at 3.0 pr. m. 4.5' Minor losses of head..... 0.7' 47.0'	Main Conduit—Salisbury Mills to City Limits. Nominal capac., 123 m. g. Actual supply, 25% less.		Popolopen Branch, and extra cost for Main Line. Nominal capac., 25 m. g. Actual supply, 23 m. g.		Little Moodna Branch, and extra cost for Main Line. Nominal capac., 11 m. g. Actual supply, 8 m. g.	
	Cost per unit	Total Cost.				
Main Line Conduit.						
26 miles C. & C. aqueduct..	$134,000	$3,484,000	21.3 m., $10,000	$413,000	22.5 m., $4,000	$90,000
3.3 miles arched-lined tunnel	308,000	1,016,000	1.2 m., 19,000	23,000	3.3 m.. 8,000	26,000
3.2 miles side-lined tunnel..	255,000	816,000	1.1 m., 14,000	15,000	3.2 m., 6,000	19,000
15 miles pipe lines and siphons..... 2 lines of pipes clear through..... 6' 8", 6' 11", 7' 2" diams.	6' 8" diam. 256,000	3,840,000	13 m., 6' 11" 32,000	416,000	14 m., 7' 2" 12,000	168,000
1.5 miles Hudson river crossing	.....	600,000	.....	50,000	.....	25,000
600 lin. ft. shaft exc.	150	90,000	.....	.....	.....	.....
26 miles R. of W. for C. & C.....	2,000	52,000	.....	.....	.....	.....
6.5 miles R. of W., for tunnel	1,800	12,000	.....	.....	.....	.....
15 miles R of W., for pipe	2,000	30,000	.....	.....	.....	.....
41 miles bridges, culverts, blow-offs, etc.....	12,000	492,000	34 m., 1,000	34,000	36 m., 500	18,000
21 siphon chambers.....	10,000	210,000	19 s. c., 1,000	19,000	21 s. c., 500	11,000
Popolopen Branch Conduit.						
Full=200' + in 3½ miles. 0.3 miles arched-lined tunnel	.....	.....	.....	50,000	.....	.....
3.2 miles cast pipe-line, 30" diam.....	.....	.....	33,000	106,000	.....	.....
3.5 miles right of way....	.....	.....	500	2,000	.....	.....
3.2 miles bridges, culverts, blow-offs, etc..	.....	.....	2,000	7,000	.....	.....
Valves, valve chambers, etc.....	.....	.....	.....	2,000	.....	.....
Little Moodna Branch Conduit.						
Full=110' + in 5¼ miles. 5.5 miles cast pipe-line, 24" diam.....	.....	.....	.....	.....	24,000	132,000
5.5 miles right of way....	.....	.....	.....	.....	500	3,000
5.5 miles bridges, culverts, blow-off, etc..	.....	.....	.....	.....	2,000	11,000
Valves and valve chambers	.....	.....	.....	.....	.....	2,000
		$10,642,000		$937,000		$505,000
Add 15% for contingencies, etc.	.....	1,596,000	.....	141,000	.....	76,000
Total cost for transportation.....	.....	$12,238,000	.....	$1,078,002	.....	$581,000
Cost of storage works.....	.....	7,972,000	.....	2,422,000	.....	681,000
Park Hill Reservoir (N. basin only).....	.....	3,696,000	.....	.....	.....	.....
		$23,906,000		$3,500,000		$1,262,000

Probable cost of the three watersheds combined into one system $28,668,000

Conclusions Regarding the Popolopen and Moodna.

The Popolopen Creek could probably be developed so as to furnish and deliver by gravity flow at 300 feet elevation inside the city limits 20 million gallons daily of excellent water at a not unreasonable cost; the conduit line would not be of excessive length, and the whole project appears worthy of accurate and complete surveys and detailed estimates.

It would, however, in its largest possible development not furnish one-tenth part of the new supply which Manhattan and The Bronx need forthwith and is, therefore, of very little weight in determining the direction of the future supply.

The Moodna Creek, so far as quantity is concerned, could probably be developed so as to furnish 100 million gallons per day, and it would probably be found possible to build a safe dam on a tight bed rock to such height that this water could be delivered to the city limits at elevation 300 by gravity flow. This can, however, not be known with certainty until test-pits and borings have been made and the geology of the valley carefully studied.

The quality of the water from the storage reservoirs that could be built in the Moodna Valley is somewhat in doubt because of the presence of various deposits of "black dirt" overlying very deep deposits of muck or peat. I have not yet had opportunity to take samples of this earth from test-pits for analysis, nor has the prospect so far appeared to warrant accurate levels for determining the depth of flowage possible over these deposits of earth rich in organic matter and whether flowage is very deep or shallow may have an important bearing. There has also not been an opportunity to study into the feasibility of covering some parts with a thin layer of gravel. So far as the facts have yet been noted they tend to cast a doubt on the quality of the water.

Comparing the probable costs of construction given in the table on page 513—which are believed to be small rather than large—with the cost of the Ten Mile river development, as set forth in Table No. 6, it is plain beyond the shadow of a doubt that the Ten Mile is a much more available, abundant and cheaper source and one more cheaply maintained than the Moodna and Popolopen.

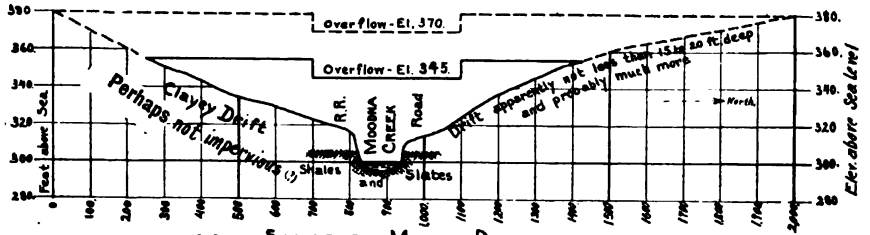
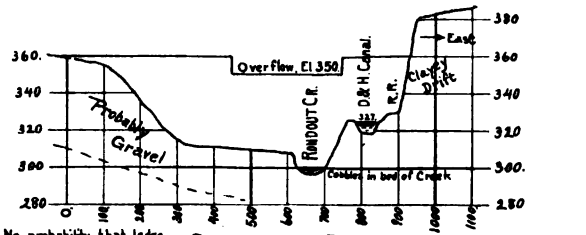


Fig. 100. SALISBURY MILLS DAM at R.R. Bridge above Village on MOODNA CREEK

While on the ground, this site was considered only with a view to the low dam, but the higher dam is a quite probable possibility.

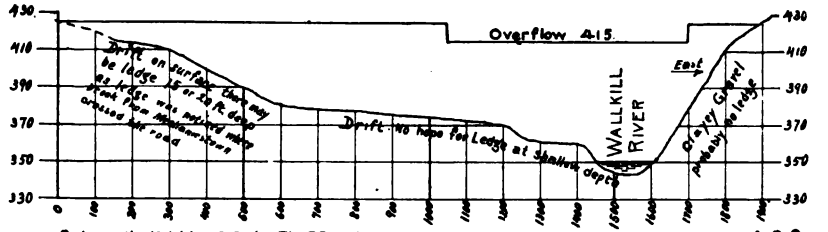


No probability that ledge can be found at less depth than 25 ft.

SANDBERG DAM. on RONDOUT CR.

Fig. 101.

About ½ mile upstream from Ellenville; Would flood say 1 sq. mile of basin above.


Overflow 415
Drift on surface, there may be ledge 15 or 20 ft. down
Drift - no hope for Ledge at shallow depth
WALLKILL RIVER
East
Clayey Gravel probably no ledge

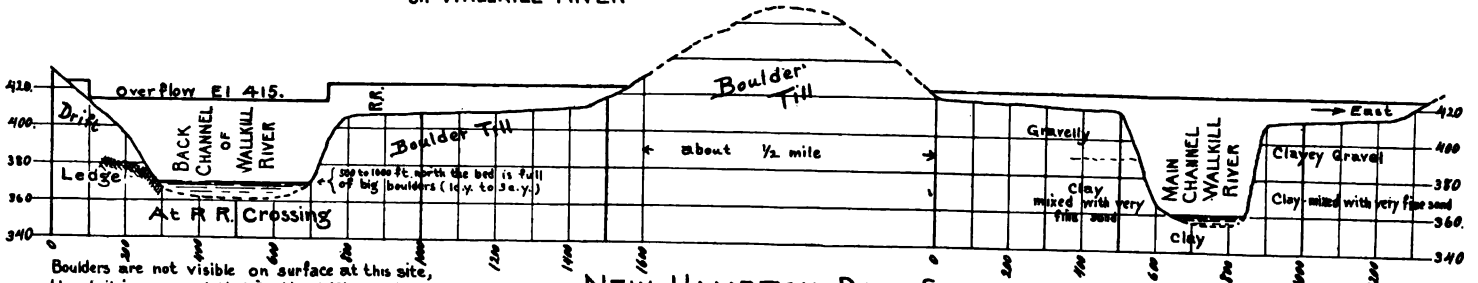
Between the Middletown & Goshen Elec. RR and Phillipsburg. Not a particularly favorable site.

PHILLIPSBURG DAM on WALLKILL RIVER Fig. 102.

Rough Sketches of CROSS SECTIONS AT DAM SITES ON WALLKILL, MOODNA & RONDOUT Crs.

From brief reconnaissance only, aided by Aneroid barometer, pacing and eye measurement; Subject to revision after instrumental surveys.

Sections made by Horace Ropes, C. E.



Boulders are not visible on surface at this site, though it is presumed that boulder drift overlies the ledge. Think it may not be very deep down to ledge

NEW HAMPTON DAM SITES Fig. 103.

Appendix No. 14.

THE WALLKILL.

The next of the possible sources, in order of proximity, is the Wallkill, which, next to the Hudson, is the largest river in southeastern New York, and therefore deserving considerable attention, although brief study on the ground shows it to be not a very hopeful source.

In elevation, it is all right for the area above Philipsburg, where it could by a low dam, be given a low-water level of 400; it has a drainage area of about 417 square miles, 210 square miles of which is in New Jersey.

The quality of the water obtainable from this source is open to very serious question.

The only opportunity or means thus far found for obtaining a storage reservoir appropriate for this large watershed, and at an elevation suitable for conveying the water to New York City by gravity flow is the somewhat startling one of reflooding the enormous drained swamp known as the "Drowned Lands." This tract of land, according to Dr. Heinrich Ries, Assistant State Geologist (see Report on Geology of Orange County, 1895, p. 474), covers 17,000 acres in Orange County and extends over into New Jersey, where I find, according to the State topographical maps, it covers 800 acres more—a total extent of swamp of about 40 square miles, all lying very nearly level, and most of which is said to have lain wet and almost worthless until drained by an artificial canal about fifty-five years ago (see Geology of Orange County, p. 466). Large areas in the upstream portion have not yet been drained or cleared from trees and underbrush.

From the Report on Water Supply, forming Volume 3 of the Geological Survey of New Jersey, page 147, it appears that to reach a part of the river that is unobjectionable for purposes of water supply, one must go back to above Franklin Furnace, in New Jersey, where the watershed is only 31 square miles, while on the other branch, the Papakating, above Deckertown, the drainage area is only 20 square miles. This upper part of the river is essentially a highland stream, but in New Jersey, ten miles before it enters New York State, it begins its course through broad, flat meadows and swamps, the head of the great area known as the "Drowned Lands." After entering New York it flows 15 miles farther through these broad, swampy flats, and beyond them enters a broad, shallow valley, the width of which increases as we go northward. The river here occupies a secondary valley eroded from the drift. This secondary valley may be described as in general 500 to 2,000 feet wide in the bottom, and bordered by terraces from 30 to 60 feet high. After flowing northward parallel to the Hudson for 60 miles in New York State it empties into the Hudson at Rondout.

When personally inspecting the stream I was much surprised to find so small a flow from so large a drainage area, near the New Hampton and the Phillipsburg dam sites, and the flood marks left on the bushes and on the margins by the recent spring floods showed a surprisingly small flood volume in comparison, for example, with the Croton.

There is an excellent opportunity to make an approximate estimate of the flow near the railroad bridge over the new channel east of New Hampton, and certainly the flow was very small in comparison with that of other streams of known drainage area at about the same time. This leads to a strong suspicion that much water is escaping by percolation through a deep bed of

porous drift—for no ledge shows here—and that it would continue to escape past any dam built near this point. This water must re-appear at some down stream locality, wherever the true rock bottom of the valley is exposed.

From the geology of the valley it would appear that the river just below the flats occupies a secondary valley formed by the erosion of drift, but that the bottom of the basin itself must be impervious. The deep bed-rock is said to be limestone and this valley is much lower than those on either side, and between it and the Hudson are thick, high walls of the crystalline rocks. At the highway bridge, south of the head of the "canal," the river runs over a bed of what certainly looks like a fine, impervious clay, having its top five feet below the level of the swamp and say five feet above the present water level. At some points this looks more like fine silt than true clay.

The ancient lake is supposed to have been dammed by a deposit of drift and probably it is through this deep and wide bed of drift that percolation occurs. Certainly this question of loss by percolation would need to be carefully investigated and accurate gaugings of the stream-flow continued for at least a year before one could be certain of obtaining the ordinary rate of yield per square mile.

This possibility of the dissolving out of cavities in the lime-stone bed rock by the acid swamp waters, must be considered in its relation to the dam-site.

REPORT OF RECONNAISSANCE.

Under instructions to follow the stream and find the best that it offered in volume of flow, favorable dam sites and points of diversion, so far as these could be learned by a rapid reconnaissance, Mr. Ropes spent about four days following the stream from New Paltz to the Jersey line, a distance along the stream of about 45 miles, driving most of the way along the highways nearest the river, but tramping along the stream for a closer inspection at what appeared to be the critical points. Long days were made and as careful and thorough an inspection as possible in that length of time. A line of aneroid barometer levels was carried along and the time of each observation noted and corrected for the hourly fluctuation by means of an excellent recording barometer stationed at the hotel. The elevations of prominent stations on the railroad profile had been previously obtained. I subsequently made brief personal examination of a part.

A number a narrow places were observed where dams 40 to 50 feet high and 700 to 1,000 feet long on the crest might be located; but below Phillipsburg these sites would either give insufficient storage or be too low to afford a gravity supply giving 300 feet head at the city limits.

The main chance to get a large supply from the Wallkill river was very obviously to dam the stream either at New Hampton or Phillipsburg and flood the Drowned Lands to a depth of about 35 feet at the lower end of the broad flats.

With its flow line at elevation 415 the area of the full reservoir would be about 55 square miles and if drawn to 390 probably about 30 square miles. The storage capacity between 390 and 415 would probably be not far from 225 billion gallons, but may be more or less. Deduct from this say 50 billion gallons to allow for amounts held back by low dams to cover shallow flowage and the available storage capacity can be safely set down as 175 billion gallons, which gives 420 million gallons storage per square mile of gross area. The water surface would be not far from 55 square miles with reservoir full (of which about 13 square miles would be in New Jersey). The per cent. of water surface to gross area would be about 13 per cent.

By diagram No. 48, on the conservative Sudbury basis the safe yield if the outlet dam and the subterranean strata were impervious should be about 850,000 gallons per square mile per day x 417 = 354 million gallons per day, but from the suspicions aroused by personally noting the small flow and the possibilities of a pervious dam site in this drift formation I should not want to call it more than 250 million gallons per day prior to further careful exploration and gauging, but on the other hand the very extensive deposits of drift throughout all this region would tend to a very large ratio of ground-water storage and a slow discharge of the rainfall into the stream.

These "Drowned Lands," which appear to be the only adequate reservoir site on the Wallkill, were once the bottom of an ancient lake bed, and are described by Dr. Heinrich Ries in his report on the "Geology of Orange County" as follows:

"These swamps occur not only in the limestone region, but also in many parts of the slate area and form perhaps the most important agricultural feature of the county. The rich black soil of the swampy tracts is enormously productive, and some of it is worth $300 an acre. This soil is generally planted with onions, and 700 bushels per acre is not an uncommon yield.

Potatoes or corn are gènerally planted in alternate years to relieve the soil. There are about 40,000 acres of swamp land in Orange County. The largest of these areas is the Drowned Lands in Warwick, Greenville, Minnisink, Wawayanda and Goshen townships, and covers 17,000 acres. Until about sixty years ago the area was covered by several feet of water held in by a dam of glacial drift at the north end. A canal cut through this dam has redeemed the land. From the Drowned Lands there arise islands of limestone or drift, which are named Pine, Great, Pellets, Gardner's, Merritts, Cranberry, Black Walnut, Fox and Seward's Islands. * * * Black soil underlies the surface to a depth of from five to fifty feet, and this, according to Mather, is, in turn, underlaid by marl. The Wallkill river follows a winding course along the western side of this area, and submerges it entirely during the spring floods."

From such hurried and scanty observations as I was able to make, I should say that much of the northern portion inside our flow line, and nearly all of it on the west side of the river, was very likely free of the deep black soil deposits. These flats were mostly open grass land, which appeared to be a rather thin covering of soil over a clay deposit. These clay beds were very clearly exposed at the river crossings near the head of the canal and on the Denton road. Southward from the New Jersey line, so far as I could see, and on the east side of the river in New York, there appeared to be considerable areas of timbered swamp with black muck. On the east side of the river also extensive areas of the black soil lands were free from timber, and were in part subdrained and cultivated.

In that portion of the Wallkill Valley southward from Montgomery, the lower side slopes appear generally to have a heavy covering of drift material.

Dr. Ries in his report on the "Geology of Orange County," page 405, says: "In the larger valleys, as those of the Neversink, Shawangunk and Wallkill rivers, the gravel and sand accumulations are of great extent and depth. Those of the Neversink valley are in places known to be over 100 feet thick;" and again, on page 463, "The valley of the Neversink river is filled to a considerable depth with gravel, which forms a broad, flat bottom and with its covering of loam produces an admirable farming land. A hole was drilled through drift at Port Jervis to a depth of 113 feet without striking bottom. Boulders were abundant in the upper thirty feet, but below that the material was mostly quartz sand with an occasional boulder. Similar gravel accumulations occur in the valley of Shawangunk kill, along the eastern base of the Shawangunk mountain. The modified drift here partakes of the nature of hillocks, whose summits are all at about the same level of 900 feet, A.T. The intervening depressions are often occupied by ponds." * * "Around the edge of the Drowned Lands, the gravel rises in rounded knolls sometimes to the height of 80 feet. There is also a conspicuous series of kames around Campbell Hall, north of Goshen. Gravel hills also line the sides of the Wallkill valley."

These extensive gravel deposits will certainly have a very appreciable effect in increasing the ground storage of this district, also towards increasing the storage capacity of the great reservoir suggested.

Although a large portion of the Wallkill watershed lies in New Jersey, its situation on the northern slope of the Highlands renders it inaccessible to the larger cities on the south side; and, as there are no places within the watershed which are likely to ever make any large draft on its waters for their own supply, New York City would be practically assured of the full flow from this area. Middletown draws its water supply from a reservoir on a brook northwesterly from the city. Goshen, I was informed, takes its water supply from Glenmere Lake. These last named sources lie within the area given as tributary to the Wallkill, but the amounts thus taken off are inappreciable.

Population.

This watershed is almost exclusively a farming district with about thirty villages ranging from 100 to 300 population by the Census of 1890; and as many more small centres of population of probably less than 100 inhabitants. The cities and towns which had more than 500 inhabitants in 1890 are:

	U. S. Census.	
	1890.	1880.
In New York—		
Middletown	11,977	8,494
Goshen	2,907	2,557
Warwick	1,537	1,043
Florida	560*	....
In New Jersey—		
Deckertown	993	370
Franklin Furnace	913*	....
Hamburg	519	412
Ogdensburg	565*	....
Sparta	501*	....

From such information as I have, I should think the outside population on farms might average about 20 to the square mile. The increase in the total population of Orange County from 1880 to 1890 was at the rate of 11 per cent.

The danger of sewage pollution from some of the larger centres should not be disregarded, although dilution, subsidence, bleaching and long storage with filtration could be relied on to in great part correct this evil.

Quality of Reservoir Bottom.

I have personally made some brief examination of the Drowned Lands and of the possible dam sites at Phillipsburg and New Hampton, and find the outlook for obtaining water of suitable quality from this source in quantity commensurate with the area, not at all encouraging, and am led to believe that the Wallkill cannot be safely reckoned among the available sources until after extended research on the practicability of purifying by filtration water that has been stored with shallow flowage over a broad and deep deposit of muck and peat.

Ten square miles and perhaps more of the 50 square miles that this reservoir would flow, are in a vast deposit of peat of unknown depth.

Some of the farmers call it "bottomless," others say 50 feet, others tell me they have run poles down 25 feet without reaching bottom. I myself ran down a 7-foot pole in a dozen places, scattered over a wide area, and it could be pushed down just about as easily as it could be pulled up. I finally left this pole pushed down to 10 feet depth in the bottom of a small test pit in the midst of the wooded swamp, nearly a mile in width, nearly a mile south of the New Jersey lines.

In the middle of the cultivated swamp west of Florida it was noted on pushing the pole down in the drainage ditches that a generous bubbling of gas (probably marsh gas) came up through the water.

In the wooded swamp one-half mile north of Pine Island the water was noted as brown in color. A sample taken from the running brook, tested, gave a color of 1.80, which is not very

* Not obtained from U. S. Census.

bad for swamp water. The color of the Wallkill at the outlet of the Drowned Lands is not so brown as might be expected; it has a slight clayey tinge which may obscure the swamp color. It has been suggested to me that the limestone sand and the organic acids may counteract each other to some degree in their effect upon the water.

I dug six or eight small test-pits 3 to 5 feet deep in representative localities in the Drowned Lands, and found the top foot with more grain and less fibre than the portions below. At a depth of about 2 feet the brown color and fibrous structure of peat were well developed in each case. Samples were collected in clean, glass jars and sent to the Analyst of the Metropolitan Water Board at Clinton, Mass., for examination by the same method (of ignition) followed in the hundreds of tests of earth now being made in connection with the stripping of the bottom of the Wachusett Reservoir, for the purpose of removing organic matter in order to ensure freedom from bad tastes in the water.

The standard adopted by the Metropolitan Water Board for the Wachusett Reservoir is to strip off or cover up all soil containing more than 4 per cent. of organic matter, and a good deal is removed that contains only 3½ per cent. (Their ordinary surface soil contains about 10 per cent.) This soil from the Drowned Lands is about 80 to 90 per cent. organic matter.

The report of tests is as follows:

REPORT ON THE ORGANIC MATTER IN TWO SAMPLES OF SOIL FROM THE DROWNED LANDS OF THE WALLKILL.

Test Pit.	Depth from which Sample was Taken.	Locality.	Organic Matter.	Per cent. Stone Remaining on No. 30 Sieve.
No. 1A	1 ft. down.	W. bank of river at highway bridge S. of head of canal	69 per cent.	None.
" 1B	3 " "	.....	85 "	"
" 1C	5 " "	.....	89 "	"
" 2A	6 in. "	1¼ miles N. 50 W. of Florida, about 300 ft. from edge, near house of Mr. Powers	*49 "	12
" 2B	3 ft. "	.....	*85 "	None.
" 3A	14 in. "	At side of Swamp road 1¼ mile N. 60 W. of Florida. About in center of that area of swamps.....	*78 "	"
" 3B	3½ " "	.....	*87 "	"
" 4A	Surface soil.	(Not manured) on N. side "Goshen Turnpike," about ¾ mile S. W. of Quaker Creek, between Big and Black Walnut Islands	81 "	"
" 4B	3½ ft. down.	.....	90 "	"
" 5A	6 in. "	N. side of road 150 ft. W. of Owen's Station, in New Jersey.....	*90 "	"
" 5B	3½ ft. "	.....	*89 "	1

The samples marked * have little or no iron present.
Analyzed by J. B. Gilmore, May 22, 1900.

Dam Sites at Phillipsburg and New Hampton.

As already stated, the brief reconnaissance by Mr. Ropes from New Paltz, upstream along the Wallkill to the New Jersey line, discovered no reservoir site sufficient to develop anything like the full yield of 400 or more square miles of watershed of the Wallkill, except the Drowned Lands, and no better dam sites for this purpose were found than those at Phillipsburg and New Hampton, neither of which can be regarded as favorable.

At the Phillipsburg site no outcrop of bed rock was found, and knowing the great depth of the modified drift deposits reported in this valley, the depth to which one might have to go for an impervious dam and for a secure foundation to hold back so vast a volume of water and to safely carry off the flood waters from more than 400 square miles, becomes a very serious problem, and of itself might forbid the execution of the plan. Extensive borings under the best geological supervision would be needed before any intelligent estimate of cost could be given, or before the best site could be picked out, although the dam would not be high above present surface.

At New Hampton two channels would need be closed—the new artificial channel or "canal" and the old or "back channel." Ledge appears in the bed of the old channel, but nothing indicating nearness to ledge appears in the new channel. The whole territory available for dam sites is apparently deeply covered with drift, and in places the gravel is clayey, but is not of "hard pan" or cemented material.

As we are now looking for all the possible advantages of this Wallkill source with a view to finding if there is enough that is hopeful to warrant further study and complete surveys, it may be suggested that since the dam will not exceed 55 or 70 feet in height from present surface of ground to the flow line some considerable percolation could be tolerated and perhaps picked up in a thoroughly filtered condition halt a mile, more or less, below the dam site by some such means as suggested in Fig. 110 for collecting the ground waters of Long Island, and pumped into the aqueduct. The dimensions of the aqueduct will therefore be figured for the full yield appropriated to a watershed of this area.

Possible Wallkill Conduit Lines.

Mr. Ropes makes the following notes on possible conduit lines. These are made up mainly from the maps and were not reconnoitered in the field, and are presented as approximations for the purpose of preparing a rough preliminary estimate for the purpose of forming an opinion as to whether the Wallkill project warrants accurate surveys.

The best location for the conduit would probably be to put the intake at Phillipsburg, and thence swinging slightly to the northward, cross the ridge between the Wallkill and the Otterkill in the vicinity of LaGrange in tunnel; thence, passing southward of Campbell Hall, the conduit would follow the valley of the Otterkill and Moodna creeks to Salisbury Mills. From Campbell Hall to Salisbury Mills could probably all be made cut-and-cover aqueduct. From Salisbury Mills to city limits the conduit would follow the route already described as a probable location from Moodna creek.

If it were possible to locate the filtration works at some point between Salisbury Mills and city limits, it would then be possible to utilize about 9 miles of the natural channel of Otterkill and Salisbury Mills reservoir, making an open waterway for that distance. With this latter condition, it also might prove better to locate the intake in the arm of the Wallkill reservoir, westerly from Goshen, and by tunneling under that city strike the Otterkill at a nearer point, and so get a greater length of open channel. With the filtration works located below Salisbury Mills it would be permissible to combine the waters of Wallkill and Moodna and carry them to the city by one large aqueduct, thereby materially lessening the cost of transportation for the two supplies. While this is a point that should be investigated farther, it appears hardly practical without the introduction of a pumping station to regain the loss of head which would be due to the filters.

Possible Short Line from the Wallkill by Tunnel.

Another possible location for a conduit from the Wallkill Reservoir, and which might be say 13 to 15 miles shorter than the one above described, would be to put the intake near Milford, and then by a tunnel about 17 miles long pass under the Highlands to the valley of the Ramapo river. Such a tunnel could probably best be built by diverging to the north of a direct course, thus passing nearly under Warwick and Bellvale, and to the north of Greenwood, Sterling and Tuxedo lakes to an eastern portal at some point in the valley near, or southerly from, Tuxedo Station. On this location the depth of shafts would not exceed, and generally it would be rather less than, 300 feet. The length of the drifts between the deeper shafts would, however, be about 2 miles. From Tuxedo the line would follow the Ramapo Valley to Suffern, and thence running southeasterly, generally about one-half mile northerly from the New Jersey boundary line, cross the Hudson river between Hastings and Dobb's Ferry, and thence to Park Hill Reservoir by the most practicable route. Between Tuxedo and city limits the conduit might be made partly cut-and-cover and partly tunnel, but probably the greater portion would be steel-pipe line. I should judge about the following lengths of different classes of work might be obtained on this route, viz.: tunnel, 21 miles; steel pipe, 17 miles; cut-and-cover, 9 miles; and open channel 3 miles. The open channel would be built from New Milford to deep water in the reservoir.

If this southern route were followed, the filtration works would be located on the east side of the Highlands, perhaps near Suffern. This "Short Line" would not permit of bringing the Moodna supply in the same aqueduct, neither would it be favorable for connection with works for intercepting and recovering the water, if any, lost by percolation through the deep and broad deposits of porous drift at the proposed dam sites.

Filtration Necessary for Wallkill Water.

The uplands bordering the Drowned Lands give the impression of an admirable gathering ground for good water (except so far as some slight hardness might result from the presence of limestone particles in the pervious soil), but the quality of the bottom of the basin, as just previously described, is such as to certainly demand filtration for any domestic supply derived from the source, and it is in the present state of the art by no means certain that filtration would work a perfect cure for all the troubles likely to arise. The growths of algæ and other low forms of life over such vast deposits of organic matter in shallow flowage under the summer sun and with the uncovering of some square miles of these flats when the reservoir is drawn down, would without a doubt occasionally, if not uniformly, be so luxuriant as to rapidly

clog filters, or otherwise interfere with their efficiency or economy of operation.

Taking for the time being the most favorable view possible in order to see if this watershed warrants any further study in comparison with the others available, we must surely include as necessary to insure fair quality of water an expense as great as that for slow-sand filtration, like that proposed for the Hudson river water at Poughkeepsie.

Shawangunk Watershed.

This drainage area is manifestly best developed in connection with the Wallkill. The extent of territory found lying at suitable elevation for gravity flow to elevation 300 was a disappointment.

The Shawangunk Kill flows through a valley at the base of the eastern slope of the Shawangunk Mountain. It is generaly parallel to and about 7 miles westerly from the Wallkill, into which stream it empties, about 3 miles south of Libertyville. Apparently, the quality of the gathering ground is excellent and the quality of the water good. The difficulties are that much of the watershed lies too low for delivery at elevation 300 in New York without pumping, and that deep deposits of drift may make the tightness of dam sites doubtful.

I have not found time for a personal examination, or for extended study through exploration and aneroid levels by Mr. Ropes or other assistants, and while the following covers the probabilities it is not exhaustive.

Mr. Ropes makes the following report of his brief reconnaissance:

"I followed this valley from Bloomingburg to Ulsterville. The drift formation which covered the country appeared everywhere to be very deep. It was generally gravel of a cleaner and more pervious character than in the Wallkill valley. In some places the bottoms were 2,000 to 3,000 feet wide; in others, the bluffs closed in, offering narrow places for dams 40 to 50 feet high, andf rom 600 to 1,000 feet long on the crest. Several such sites were observed, but I did not see any which looked especially attractive, because of the prevalence of coarse drift in the bluffs and bottom.

I only made a note of the site at Bloomingburg, which is probably as good a point as any for diverting water for delivery at elevation 300.

At this site a dam 50 feet high and 1,500 feet long on the crest is perhaps practicable, without interfering greatly with the village. The bottom length of the dam would be about 300 feet. The bluffs, each side, are apparently all clayey gravel. There were boulders in the bed of the creek. The basin above the dam site looked large, and I should say favorable for flooding at least 1½ square miles with a 50-foot dam. The brook was 50 feet wide and the water had an excellent appearance.

Reference has already been made to the diversion of this stream in connection with the supply from the Wallkill. The proposition was to divert its waters at Bloomingburg, by a small dam and about 8 miles of tunnel through the dividing ridge, into the Wallkill Reservoir.

The area of the watershed above Bloomingburg is estimated from the map to be.....47 square miles.

This, with 5 per cent. of water surface at 750,000 gallons per square mile of gross area per day, would yield a supply of..... ..35 million gallons per day.
The reservoir capacity required to maintain this rate of yield during the cycle of years of smallest rainfall would be, with generous ground storage, not far from..... ..... .9 billion gallons.

This storage can be readily and cheaply provided in the proposed Wallkill Reservoir, and it is believed to be possible to so proportion the works for the Shawangunk that but little storage need be provided on that stream, but its combination with the Wallkill water would require the expense of its filtration.

If the storage of the Shawangunk water on its own grounds were found to cost $220 per million gallon capacity, or $2,000,000 in all, the interest and other expenses on account of this storage would amount to about $100,000 per year, or probably at least $8 per million gallons, and as the proposed Wallkill Reservoir capacity is more than ample, it might prove about as cheap to store it there and afterward filter it as to build reservoirs on the Shawangunk and avoid filtration.

For present purposes, assuming the storage of the Shawangunk water in the Wallkill Reservoir, the problem has been treated in the following manner :

Water level in creek at Bloomingburg is about elevation 435.

Flow line of Wallkill Reservoir elevation 415.

Hydraulic grades of tunnel, say 455, 414, 41 feet fall or for 8 mile tunnel, slope 5 feet per mile.

It is estimated that a lined tunnel, 6 feet diameter in the clear, would have a capacity of 90 million gallons per 24 hours with this slope. The excavation, to the prescribed line, would have a diameter of 7.3 feet, or a size about as small as can be conveniently worked.

It is probable that the flood discharges from this watershed rarely exceed 25 cubic feet per second per square mile, and if the entire discharge of a single flood be considered equivalent to that rate, continued through 24 hours, the total amount would be 47 square miles by 25 cubic feet per second by 7.48 by 86,400 seconds—761 million gallons per 24 hours. The tunnel would in that time carry off somewhat more than 90 million gallons, so that, roughly speaking, about 670 millon gallons temporary storage would be needed to equalize this assumed flood.

If at this dam site a 40-foot dam should be built, with the overflow crest at elevation 465, it would probably give a reservoir of about ⅓ square mile, which, for an average depth of 10 feet, would have an available capacity of 700 million gallons.

PROBABLE COST OF WALLKILL RESERVOIR.

(A very rough approximation from hasty reconnaissance; taking the most moderate values that appear reasonable.)

Cost of main dam, including foundations, deep core wall, temporary work, gate house and intercepting gallery, etc., complete, probably at least.....	$1,500,000
Two low dams to cover shallow flowage, say 4,000 feet by 10 feet.....	250,000
Land damages, 2,000 acres at $300.....	600,000
Land damages, 39,600 acres at $100..... This does not allow for the contingency of extra expense on the New Jersey flowage of about 13 square miles.	3,960,000
Dwellings and other buildings (note that the flow line would encroach on borders of several villages, notably Mechanicstown, New Hampton, Denton, Deckertown, Milton, Glenwood, Pine Hill, Florida and Orange Farm), say at least.	1,000,000
Damages at Pine Island granite quarries, miscellaneous contingencies, removal of cemeteries, legal expenses, etc.....	500,000
Grubbing and clearing 3,000 acres of wooded swamp, at $70 per acre.....	210,000
Relocation of railways, 30 miles at $18,000 per mile	540,000
Relocation of highways, 40 miles at $12,000 per mile.....	480,000

The separation of the populous farming regions of Orange County by this great lake would doubtless raise great opposition, unless some highway were provided across it near the New York State line, therefore add for	
Embankments and bridges across reservoir, for highways, 4 bridges and embankments, say 19,000 feet total length by 30 feet high and rip-rap, perhaps.....	$1,000,000
Caring for sewerage of Middletown, Goshen, Deckertown and Warwick, say....	200,000
Damages to water powers at Phillipsburg, Montgomery, Walden, Wallkill, Galesville Mills and elsewhere, including legal expenses and allowance for undeveloped power, say at a guess.....	500,000
	$10,740,000
Add 15 per cent for contingencies and supervision.....	1,610,000
Total cost of water rights and works for collection and storage, say, very roughly.....	$12,350,000

PROBABLE COST OF SHAWANGUNK WORKS.

(A very rough approximation from hasty reconnaissance.)

Cost of diverting dam, say.....	$150,000
Land and buildings, 300 acres, at $150.....	45,000
8 miles right of way, 50 feet wide, for tunnel, and including possible damages for draining meadows, springs and wells, at $1,600 per mile.....	13,000
Relocation of highways, 2 miles, at $12,000.....	24,000
Connecting tunnel, 8 miles, slope 7 feet per mile, diameter inside of lining 6 feet; cost per mile, $153,300.....	1,226,000
750 linear feet shafts, at $75.....	56,000
	$1,514,000
Add 15 per cent. for contingencies and supervision.....	227,000
Total (very roughly).....	$1,741,000
With the past rate of increase in water consumption these works would not be needed for 18 years, which would make their present value, with interest, 3 per cent..	$1,023,000
Or in round numbers, say, at least.....	1,000,000
The above supposes Shawangunk water stored in Wallkill Reservoir. If stored separately on Shawangunk to avoid filtration expenses, add to present value, say.....	2,000,000

PROBABLE COST OF WALLKILL FILTRATION WORKS.

(A very rough approximation, made without definite locations.)

To make this as favorable as possible under any reasonable view, assume that of the Wallkill water only 250 million gallons has to be filtered, the remainder being collected as ground water* below the dam. Add from Shawangunk 35 million gallons. Total daily, 285 million gallons to be filtered.

The cost of this slow sand filtration plant is estimated on the same basis (except for some slight reductions on account of easier work in preparing the ground) as the Poughkeepsie project.

285 million gallons + 25 per cent. = 356 million gallons total capacity; built 100 million gallons at start and remainder later when required.

* It is probable that the construction and operation of intercepting galleries and pumps would make this ground-water portion of the supply about as expensive as that drawn directly from the pond and filtered; but to favor the problem neglect this possible expense.

100 million gallons plant, with all lands and buildings required for whole plant, at $23,500 per million gallons, about	$2,350,000
Filters with a capacity of 100 million gallons would probably cost (exclusive of land and buildings) $22,500 per million gallons, or $225,000.	
Present value, with interest at 3 per cent., of additional filters, with capactity of 100 million gallons per day, to be built 8 years later	1,780,020
Present value of 156 million gallons plant, deferred for 15 years	2,250,000
Total	$6,380,000

PROBABLE COST OF WALLKILL CONDUITS.

(A rough preliminary approximation based mainly on a study of the maps without reconnaissance over the line.)

Length: 63 miles from Phillipsburg to City Limits.
Capacity: A mean delivery of 400 million gallons per day, plus 25 per cent. excess = 500 million gallons per 24 hours' capacity when clean.

37 miles of cut-and-cover aqueduct, at	$211,000	$7,807,000
4.8 miles of lined tunnel, at	405,000	1,944,000
4.7 miles of side-lined tunnel, at	332,000	1,560,000
15.0 miles pipe line and siphons, 2 lines, 8 feet 3 inches diameter (½ inch) each, at	157,000	4,710,000
1.5 miles Hudson river crossing (part steel-pipe line and part steel-lined tunnel)	.....	900,000
15.0 miles 3d line of pipe deferred 12 years	.....	1,652,000
15.0 miles 4th line of pipe deferred 16 years	.....	1,467,000
800 linear feet shaft excavation, at	150	12,000
37 miles of right of way for cut-and-cover, at	3,000	111,000
9.5 miles tunnel right of way, including dumps and shaft sites, at	4,100	39,000
16.0 miles right of way for pipe line and siphons, 90 feet wide, at	2,700	43,000
53 miles bridges, culverts, manholes, blow-offs, etc., at	21,000	1,113,000
19 siphon chambers, at	14,000	266,000
		$21,624,000
Add 15 per cent. for supervision and engineering.	.....	3,244,000
Probable total cost of transportation works	.....	$24,868,000

Water-Power.

Dr. Ries, in his report on Geology of Orange County, 1894, says:

"The Wallkill is a sluggish stream with little fall until it reaches Walden and Montgomery, at both which points it cuts through the Hudson river slates. Two mills and a paper factory are in operation at Montgomery, where the fall is increased by a dam across the river."

"At Walden the river has a natural fall of 26 feet, but since the dam above the bridge was constructed a fall of 32 feet is obtained. This fall is utilized by the New York Knife Company, employing 350 hands. The water passes through six turbine wheels, which generate 195 horse-power." * * * * "The Wallkill Valley Knife Works below the bridge receive 60 horse-power from the river."

Below this I noticed a dam about 15 feet high furnishing power to a paper mill at Wallkill, and a low dam at Galesville Mills.

The U. S. Census of 1880 shows that on the tributaries of Rondout Creek, in Orange County (and which must apply principally to the Wallkill river), there were 41 mills using 1,147 horse-power. This Census Report gives no data about the Wallkill in detail.

How much of this is actually affected by the proposed diversion of the Wallkill waters I am unable to state, but considering what is now utilized at Phillipsburg, Montgomery, Walden, Wallkill and Galesburg Mills, together with the possibilities for greater development, it appears proper to include in our rough first approximation an allowance for 800 developed, and, say, 600 undeveloped horse-power being taken away by the proposed diversion; this figure being, however, more of a guess than an estimate.

A ROUGH APPROXIMATION TO COST PER MILLION GALLONS OF POTABLE WATER FROM WALLKILL AND SHAWANGUNK.

400 million gallons daily × 365 = 146,000 million gallons per year.

(There is much doubt if a safe yield of 400 million gallons, or of 365 million gallons from Wallkill only, could be obtained because of possible loss by percolation at dam site; but to give the project the benefit of the doubt pending more complete survey the full theoretic yield is assumed.)

Construction.

Cost of Wallkill Reservoir, about		$12,350,000
Cost of Shawangunk Works, exclusive of Wallkill storage ($1,741,000), deferred 18 years, about		1,000,000
Cost of Filtration Works, 100 million gallons' capacity, built now.	$2,350,000 00	
Cost of Filtration Works, 100 million gallons' capacity, deferred 8 years	1,780,000 00	
Cost of Filtration Works, 156 million gallons' capacity deferred 15 years	2,250,000 00	
Total present value of cost of filters		6,380,000
Cost of conduits in part deferred		24,868,000
Cost of Park Hill Reservoir (North Basin only)		3,696,000
Total		$48,294,000
Total cost of works for collection, storage, filtration and transportation for 400 million gallons' daily supply, not less than		$45,000,000

Operating Expenses.

Collection and Storage.	Total Cost per year.	Per Million Gallons.
Interest on entire cost, structures and land for collection and storage, 3 per cent. per year on $12,350,000+$1,000,000=$13,350,000, about	$400,000	$2 74
Sinking fund to pay off cost in 40 years at 3 per cent. interest, 1.326 per cent. per year on $13,350,000	177,000	1 21
Taxes and special assessments at 0.4 per cent. entire cost	54,000	37
Operating expenses and repairs and maintenance, say	60,000	41
Extraordinary repairs and depreciation (kept good from maintenance)	.....	....
Total cost collection and storage	$691,000	$4 73

Filtration of 285 million gallons daily from Drowned Lands × 365 = 104,000 gallons per year.

	Cost of Filtering per Million Gallons Filtered.	Total Cost per Year.	Cost of Filtration Spread over Entire Quantity Delivered.
Interest on entire cost of land and structures (reduced to present value), at 3 per cent. per year	$1 84	$191,000	$1 31
Sinking fund to pay off cost in 40 years at 3 per cent.—1.326 per cent. per year	84	84,000	57
Taxes and special assessments at 0.4 per cent. of entire cost	24	25,000	16
Extraordinary repairs and depreciation	96	100,000	68
Maintenance, operation, labor and supplies at $3 per million gallons, 3 × 104,000	3 00	312,000	2 14
Total cost of filtering	$6 88	$712,000	$4 86

It is quite possible that the quantity collected from percolation would cost as much for collection and pumping as the water drawn directly costs for filtering.

Transportation—400 million gallons per day x 365=146,000 gallons per year.	Total Cost per Year.	Per Million Gallons.
Interest on entire cost of structures, including Park Hill Reservoir, and land, at 3 per cent. per year	$857,000	$5 89
Sinking fund to pay off cost in 40 years at 3 per cent.—1.326% per year	379,000	2 60
Taxes and special assessments at 0.2 per cent. of entire cost	57,000	39
Extraordinary repairs and depreciation	134,000	92
Operating expenses and repairs	100,000	68
Total cost of transportation	$1,527,000	$10 48
Total of annual charges, comprising interest, sinking fund, taxes, maintenance and operating expenses	$2,930,000	.....
Total cost per million gallons, all figured on lowest reasonable basis and assuming the full quantity obtainable	20 07	.....

While the total cost per million gallons figures out low, it must be remembered that with a view to finding all reasonable warrant for the further study of the Wallkill source we have not included so many extras as were included in the Ten Mile and Housatonic estimate, and there are several very important contingencies not covered, notably the possible cost of securing the 13 square miles of the reservoir site which lies within the limits of New Jersey, and the possible extra expense of constructing a safe dam on this very doubtful foundation which, perhaps to a great depth, is porous, modified, glacial drift.

The great and overshadowing objection to the Wallkill project is the questionable quality of the water stored in shallow flowage over this vast deposit of muck and peat, and the grave doubts about filtration forming a perfect safeguard.

Springfield, Mass., after twenty years of bad water, may soon have to abandon its expensive Ludlow Reservoir, and Greater New York could not afford to become committed to any such investment as the Wallkill source would involve without the most thorough expert investigation upon the purification of water stored in shallow flowage over a swamp bottom by filtration and long continued experiments.

Rondout Watershed.

Next beyond the Wallkill and the Shawangunk lies the Rondout Creek, and from the road and township maps it would appear hopeful that much more than 200 square miles of drainage area could have its yield by gravity flow diverted into the same general aqueduct system as the Shawangunk and Wallkill.

The main objections found in our brief examinations to this watershed are:

First—Its small watershed above the elevation required for delivery to New York at high pressure by gravity flow.

Second—The limited opportunity for storage reservoirs and the consequent limitation of the yield of the watershed above Ellenville to probably somewhere between 25 and 50 million gallons per day.

Third—The great difficulty, delay and expense involved in bringing this water under or around the Shawangunk Mountain toward New York.

The maps and a brief examination of the Ramapo Company's Napanach and Ellenville sites appeared to give so little encouragement that in the pressure of other matters no detailed study of the possibilities was made in the field.

The present lack of information about the Rondout watershed furnishes another instance of the great advantage that would follow the extension of the topographical survey to this region, and of the difficulty and slowness of progress in answering questions on water supply without good topographic maps.

The head waters of the Rondout Creek are mountain streams, each draining a comparatively narrow territory, and from such brief views as were had of a few of the valleys in which these streams flow, they are typical ravine-like, narrow valleys with the stream falling rapidly, and thus presenting little expectation of economical storage sites. Because of our scant time they were not explored. These streams, flowing generally in a southeasterly direction, have their focus at Ellenville and Napanach.

The Ramapo Company has filed on two reservoir locations lying very near together and adjoining the Village of Ellenville, and has called them the Sandberg and the Napanach Reservoirs. The Delaware and Hudson Canal location is alongside these sites.

Mr. Ropes' brief examination of these sites, and the barometric and railroad levels showed both the Napanach and the Sandberg sites to be too low for economical delivery of the water at elevation 300 in New York by gravity flow.

The Sandberg site is on much the smaller tributary, but is the higher of the two. Its flow-line might perhaps be put as high as elevation 350, and allowing only 10 feet for drawing it down, we should have only 40 feet left to give slope to a conduit that would have to be little, if any, less than 80 miles long to reach New York.

The Napanach Reservoir is fed by a large drainage area, but apparently could not have its flow-line carried much, if any, above elevation 320 above sea.

We made no exploration along the different branches in the high mountains to the westward, for the outline maps available indicated little hope of finding adequate storage at a reasonable cost. Even if a suitable site could be found on each of the principal branches, there would be the disadvantage of having a number of widely separated reservoirs, on different streams, requiring expensive branch conduits to gather their water into the main line, and entailing increased expenses for operation.

Another very serious obstacle to obtaining a supply from the Upper Rondout is the difficulty of getting it out. On the east side of the valley the Shawangunk Range extends in an unbroken line from the New Jersey border, north, nearly to Rosendale. The elevation of the passes near Ellenville are about 1,500 feet above sea, and the width of the range at narrowest parts of its base must be nearly five miles. To take water from the Sandberg Reservoir to New York would require either that the conduit line should be deflected north about 15 miles to get around the end of Shawangunk Mountain, or that a tunnel must be driven through the mountain, which would not be less than 4 miles long between shafts of reasonable depth; and, because of the low elevation at which this grade line must be laid in order to drain the Sandberg Reservoir, the conduit would be practically all tunnel for nearly 24 miles to the vicinity of Newburg.

With the foregoing facts in view, although the exploration was much less complete than we would have made had there been more time, we feel safe in saying that water supply for New York City from the Upper Rondout is impracticable at reasonable cost in comparison with other sources.

Appendix No. 15.

BROOKLYN.

By the very complete estimate of cost of the proposed Long Island supply from east of Massapequa, made by Mr. I. M. de Varona in 1896 (see p. 74, Brkln. Rept.), the total cost of construction was estimated at $24,500,000, and the cost per million gallons was estimated at $39.03.

In this the interest is figured at 3.5% and the sinking fund as entered in estimate is about equivalent to a 50 year annuity at 3% which is .0088 per year. (On page 31 of Brooklyn Report Sinking Fund is stated to be 3½%, but the period is not stated. The actual entry in Table 47 shows .008805 per year for works as a whole.) The total of Mr. de Varona's interest and sinking fund are thus 4.38%, while J. R. F.'s total, with a 40 year annuity, is 3 + 1.326 = 4.33%, or substantially the same.

It is, however, to be noted that in all the Freeman estimates, while the above annuity is provided for to retire all the bonds at end of 40 years, a second annuity is added "for renewals," or to form a sinking fund to replace old pumps and boilers with new after an assumed life of 25 years, and to replace old steel conduits with new after an assumed life of 50 years, and to, in a similar manner, renew many other parts of the works, so that the annual charges in the Freeman estimates not only retire all the bonds at the end of 40 years but also turn the works over "good as new" at end of term. An allowance of 0.4% for taxes and special assessments is added, which does not appear in the Brooklyn estimates. See this report p. 332–6.

So far as I am able to analyze Table No. 47 of the Brooklyn report for the purpose of putting the above estimate of $39.03 per million gallons strictly in parallel with the estimates in Tables No. 6 and No. 8, it appears that fully $1 per million (and perhaps $1.75) would have to be added to cover this larger annuity, etc., making total cost at least $40 per million gallons delivered, if put on same basis as Tables Nos. 6 and 8.

Plainly the Housatonic will give very much the cheaper supply per million gallons, besides saving $20,000,000 of the immediate expenditure.

If the Housatonic high-pressure gravity supply proves inexpedient because of inter-State complications or extravagant demands, then all the information that I have been able to collect points very strongly to the importance of immediately developing the Long Island sources east of the Massapequa and utilizing the drainage from substantially the whole of the territory proposed in the Brooklyn Report of 1896.

SOURCES IN SUFFOLK COUNTY.

I have not had time to study this matter sufficiently to present definite recommendations or designs for structures, but have spent several days driving through this watershed, carefully examining it and noting its freedom from pollution, inquiring from the farmers the depth of their wells and the elevation of the ground water, and in company with an experienced geologist, have made in the field such brief investigation as the time afforded, of the features bearing upon the question of water supply. Everything yet seen points to this territory east of Massepequa, as an uncommonly good gathering ground for domestic water supply.

There are indications in the geological structure and in the depth of a few wells that the catchment area does not extend so far north as the surface levels indicate or as is marked upon the maps of the Brooklyn Report, but this can probably be in some degree offset by improved methods of collection.

The territory is remarkably free of sources of pollution, will probably yield an unusually high percentage of the total rainfall, and in the interstices of its loose gravel formation above the impervious floor of clay, which gravel formation extends for probably an average depth of from 40 to 60 feet over almost the entire area under consideration, there is a natural storage reservoir for retaining not only the rainfall of February in storage ready to be drawn upon in October, but of even greater storage value, in storing up the rainfall of a series of wet years so that it can be drawn upon in a series of dry years, and giving promise that it will be possible to utilize a larger proportion of the total rainfall than in the most generous system of storage in surface reservoirs or natural lakes. This subterranean storage is safe beyond ordinary pollution and shielded from ordinary evaporation loss and costs nothing for dams; it merely waits for cheap, well-considered treatment to extract its water, and I am led to believe that a moderate amount of experimental work will prove that this can be done in a better manner than heretofore and with such slight lowering of the ground water-level at any one point, that a large portion of the water can be taken away with no injury to the growing crops and with no damage or inconvenience to the farmers. If not drawn into an aqueduct this water must escape slowly and unseen into the sea, through the porosity of the deep gravels, and it would appear just as well that a part of it be drawn by an underground channel toward Brooklyn, as that all should run to waste into the sea.

The foregoing applies only to the water in the upper gravel above the blue clay. I am led by the result of these investigations to believe there is no hope of securing any considerable quantity of good water from the lower gravels by deep wells.

Supply from Deep Wells on Long Island.

The possibility of obtaining an ample supply of good water from deep wells extending beneath the upper, or blue clay, and tapping deeper water-bearing strata, has probably fewer advocates to-day than a few years ago, but it appeared proper to give the subject some study, particularly since it was conspicuously mentioned, with some hope, on page 8 of the Brooklyn Report of 1896. The success, in a limited way, of some wells on Long Island and of many wells in New Jersey, helps to make this possible source

of supply arouse popular hope. If the case is really hopeful, exploration should be extended by new test wells farther east; if the case is hopeless, then this should be found out and the illusion not be permitted to stand in the way of real progress.

As a result of the studies hereinafter described *I find no encouragement for spending any considerable sum on deep wells with the hope of obtaining therefrom any sufficient supply for municipal purposes.* There appear the best of reasons for expecting that water from the deep gravels will be more expensive to obtain than from the upper gravel, inferior in quality because of iron, sulphur and other mineral elements in solution, and the geological structure of the sub-strata appears to absolutely forbid the withdrawing of any large volume for a long period without an admixture of salt or brackish water being drawn in, or robbing the upper gravel of water more cheaply obtainable.

Whatever potable water is continuously taken out of the ground upon Long Island must have its source in rain falling upon Long Island. Deep artesian wells can by no possibility be supplied by water from the mainland.

The situation may be rounded up as follows: On the north there are no deep pervious strata and the primitive impervious bed-rock lies near the surface all along the Connecticut shore. On the east and south lie the Atlantic Ocean. On the west any possible subterranean flow through deep-lying porous strata must be cut off by the continuous dam of impervious trap rock which outcrops forming the Palisades and continues all the way south under Jersey City, Bayonne, and to Perth Amboy; a secondary dam is found in the rocky backbone of Manhattan Island and the line of serpentine rock underlying the Staten Island hills.

This matter of a possible supply from deep wells has been already explored at considerable expense to the city by certain test wells, which are doubtless worth their cost for the information and the limited quantities of water that they give. The information as to the sub-strata obtained by these borings is extremely valuable, and while for scientific purposes a few more deep wells in carefully selected locations would be worth their cost, in making sure of the geological formation over a wider extent of territory, a careful study of the geological indications found on the surface, taken with what the present wells have shown, gives no hope of adding to the water resources from tapping the deep gravels to any such extent as needed for the future supply of Brooklyn and Queens.

Stated in popular language: The blue clay stratum averaging 100 feet thick, covering nearly the entire area of Long Island, at a depth averaging,

say, 40 or 60 feet beneath the surface and immediately beneath the yellow gravel, forms, probably over most of this area, a water-tight roof sloping gently to the south over the lower gravel, which prevents the rain water from reaching the lower gravel, excepting along its narrow northern edge and through such holes in this broad sheet of clay as have been plowed out by the glacial ice cap, or furrowed out by the streams from the melting ice at the close of the glacial epoch.

Beneath this water-tight roof of clay lies a stratum of water-bearing gray gravel 10 to 100 feet thick, and beneath this gray gravel lie strata of clays and gravel alternating and aggregating 500 feet or more in thickness above the bedrock.

The spaces between the particles of gravel and sand in these strata between the roof of clay and the floor of primitive rock, say, 300 or 400 feet in total thickness, exclusive of the clay strata, form probably, 33, or at least, 25 per cent. of the entire volume of this gravel and sand and these interstices are filled with water imprisoned there for ages or very slowly percolating seaward as fresh water from rainfall comes in through holes in the roof; but any large draft upon this water must, in course of time, if larger than the inflow from the small area receiving rainfall which drains into it, slowly draw in sea water to take its place. This subterranean reservoir beneath the upper blue clay, if 25 per cent. of the entire mass 400 feet in depth is water, may be regarded as a reservoir 100 feet deep and of an area about as large as Long Island itself, and having an exceedingly slight current southward, from the small inflow of rain water.

If, to obtain a clearer conception, we must assign numerical values, let us say that, as with ordinary storage of rainfall upon the surface, a million gallons per day per square mile is the utmost average yield, then if we draw out from deep wells 25 million gallons per day, there must come into the gravel to take its place a volume of water equivalent to the rainfall drainage from 25 square miles of territory, and unless the leaks in our clay roof have tributaries of 25 square miles of area on the surface of Long Island, sea water must slowly, in the course of years, work back in to take its place.*

And if sufficient water comes in through the leaks of the sloping clay-

* A little arithmetic indicates that if our deep well is, say, 2 miles away from the outcrop of the cretaceous gravel under the sea and the material homogeneous, so that water would percolate with equal freedom in all directions, and the equivalent depth of net water 100 feet, half of which could be extracted, if the foregoing be granted, then to exhaust a reservoir of 4 miles diameter 50 feet deep would require only 4.5 years steady pumping at the rate of 25 million gallons per day. Of course the flow would *not* be *exactly* uniform in all directions and there would be *some* influx of fresh water, but this simple computation shows that these deep subterranean reservoirs are *not* inexhaustible, as has indeed been practically shown by the water at one or more of the deep wells becoming brackish.

roof to supply this draft, then the possible yield of the yellow gravel and the surface streams is robbed thereby to the full measure of this leakage, of the water it could otherwise yield through shallow wells or surface flow.

It has been suggested that deep strata might bring water from the mainland of Connecticut across Long Island Sound, and that if so, these lower gravels would furnish an inexhaustible supply. This matter has in the past been strongly presented and was indeed, as I am informed, the main argument used to encourage the boring of the present deep wells. I therefore called to my aid Professor William O. Crosby, of the Massachusetts Institute of Technology, who has given much attention to economic geology. Professor Crosby made a careful compilation of all the available publications by other geologists relating to the geology of Long Island; corresponded with certain geologists who have studied special features of this field, and together we visited the pumping station at Massapequa, examined samples from well borings, drove north across the island to Hicksville and Bethpage, then eastward to Hempstead and back to Myricks. On another day, starting at Patchogue, a line was explored across the western side of Lake Ronkonkoma, thence around the lake and eastward nearly along the line marking the southern limit of glacial action, to Coram; and thence across to the shore line and clay beds of Port Jefferson, and back via Medford to Patchogue, noting at various points by aneroid barometer the surface level and the water level as shown by wells and ponds. On a third day, the formations and outcrops of the north shore, back and forth near Oyster Bay, were studied in the field. Long days were made and many miles of road travelled each day, and the reconnaissance served to give a comprehensive view of the territory tributary to Myricks and Massapequa and of the middle portion of the territory proposed for additional supply in the Brooklyn Report of 1896, and while a much more extended and through research would be interesting and make the boundaries more sharply defined, the main facts stand forth plainly already.

Professor Crosby's report is given in Appendix No. 16, and his conclusions are that any *large* supply below the blue clay is hopeless, the good quality of water from any projected deep well uncertain; and that any water coming across from the Connecticut mainland an impossibility, and that while similar geological strata give good water in New Jersey, there is no apparent means for this water to cross to Long Island.

Thus, by elimination of deep wells, we have left for consideration only the same sources reported upon by Mr. de Varona in 1896, namely surface streams and driven wells in the top gravel.

Supply from the Upper or Yellow Gravel.

I have not had sufficient time to fully work out certain plans for developing this water in the way that appears to give the best promise of quality and quantity and would hesitate before suggesting a departure from methods and estimates which show so much care as Mr. de Varona's, but since there have been objections to diverting any of this water very forcibly brought out since the time of Mr. de Varona's studies, culminating in the refusal of the State to permit any more of the Long Island water from Suffolk County to be diverted toward Brooklyn, and since heavy claims for damages have been entered against the city for diversion of these ground waters at certain points, it is profitable to discuss briefly the conditions regarding the subterranean flow and to suggest other and perhaps better means of taking the same water.

I may venture to add that notwithstanding the experience of Mr. de Varona and his associates and predecessors, which has been to a considerable extent adverse to driven wells, I am inclined to *regard the underground water stored in the interstices of the saturated yellow gravel above the blue clay, as affording the very best of storage, ample in volume, removed from pollution, and in many ways cheaper and better than the storage to be obtained from surface ponds or reservoirs,* and would be led to seek somewhat different means of taking out the ground water than those heretofore employed. It, at present, appears to me to be preferable to tap the saturated upper gravel at a depth of, say, 40 or 50 feet, at such frequent intervals that the level of the water table would not be so much disturbed locally as by the present methods of driven wells and pumping stations a mile or so apart.

In the past, the surface streams and surface storage have stood first in favor and the ground water taken as a last resort. *I believe this order of preference can be reversed with probable advantage in safety to health and in economy of construction and in the lessening of damage claims.*

The new territory may be found free of some or all of the troubles that have brought the shallow and deep wells both into disfavor at Jameco, Clear Stream and Forest Stream, and this question of iron and other impurities in the water of the new territory could be answered in advance with tolerable certainty by inexpensive wash-drill exploration.

On page 15 of the very full and valuable description of the Brooklyn Water Supply prepared by Mr. I. M. de Varona, published in 1896, the case against driven wells is very forcibly stated as follows (see pp. 15–16):

"Driven wells, as a continuous source of supply, are open to various objections. The extreme difficulty (except in special cases) of determining the tributary area, prevents any satisfactory estimate of the probable yield, and for the same reason, in case of pollution, it is difficult to

remedy it or even trace its sources. * * * Doubts as to the permanence of the supply may well be entertained, * * * possibly also deterioration in quality. * * * Legal complications attending extent of area affected may be apprehended. * * * Infiltration of salt water from the ocean imminent, except under the most careful management."

"The water from a deep test well at Forest Stream Station is so impregnated with sulphuretted hydrogen as to be unfit for use."

"At Jameco, Clear Stream and Forest Station the iron, silica, magnesia, alum, lime and other constituents of the soil combine to form an almost impervious coating over the well strainers, greatly diminishing or totally obstructing the flow."

From the context we note—

At Jameco—The shallow wells averaging 43 feet in depth give much trouble from clogging.

At Clear Stream—The 150 wells average 38 feet in depth, as redriven in 1894.

At Forest Stream—Wells driven in 1884 were pulled up in 1894 and redriven. The average depth of the shallow wells at this station being 41 feet.

I therefore venture to suggest a different line of development and to present some of the reasons for thinking that *the ground water from shallow wells 30 to 70 feet deep, but not penetrating the blue clay, may be found preferable to a development of the surface water.*

First—True Watershed Area:

The area of the subterranean watershed can be learned in advance with sufficient accuracy to permit reliable estimates of the safe yield of wells at a given locality.

From the geological studies of Appendix No. 16 we may question if the area tributary to the Brookhaven conduit line proposed in the Brooklyn report of 1896 extends nearly so far north as indicated on Maps Nos. 1 and 11 of the said Brooklyn report, and as I have stated in the footnote to my Map No. 17, at the worst, the northern boundary of this watershed may perhaps be found on an east and west line passing just south of Lake Ronkonkoma, thus cutting off nearly 40 per cent. from the area of 200 square miles (about 235 by scale of maps) indicated as available in the Brooklyn report; this non-conformity of the surface divide with the subterranean divide being due to the pushing up the clay stratum into a continuous ridge, of which the Bethpage clay-pits give some intimation, by the advance of the great continental glacier.

In my line of barometric levels across the island from Patchogue to Port Jefferson I noted a very suggestive ridge extending easterly between Medford and Coram, and found the ground water level near Port Jefferson as shown by wells apparently sloping to the north while the surface of the ground sloped south. This is a matter of extreme importance and demands very complete study.

Whether or not there is actually so small a gathering ground as only 120 square miles instead of 235 or 200 could be learned by a series of com-

paratively cheap and shallow wash-drill borings, made under proper supervision, and by observations made once a month at these wells upon the elevation of the ground water in the yellow gravel.

Water flows "down hill" in percolating through porous gravel just as certainly as on the surface, and by determining the elevation of the ground water and plotting its contours, much as Mr. Kirkwood determined it in some of his earlier studies on the Brooklyn supply, the direction of flow and the true limit of the watershed could be made known with certainty.

Second—Slow Transportation of Ground Water:

The freedom of the subterranean water from pollution and the length of its time of transit in the ground, during which bacterial life would most probably cease, is indicated by a little study of its course while slowly flowing through the gravel. Figure No. 104, not drawn to scale, may be considered as representing a north and south vertical section along the line of flow. Assume the total length from the watershed line to the conduit line to be 8 miles and the depth of the saturated gravel 50 feet. These dimensions are not far from the actual distance and probable depth on a section near Massapequa.

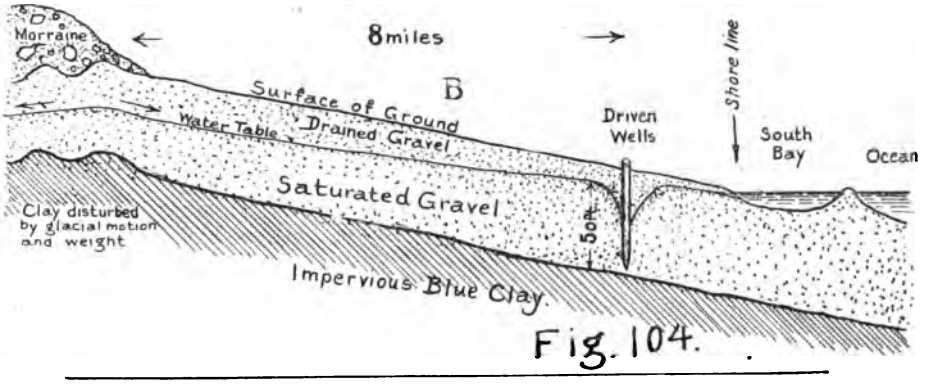


Fig. 104.

Numerical values will serve to give definiteness to our conceptions and with these round numbers as a basis the results can be reduced to fit the width and depth at any particular locality.

The mean velocity of the ground water flowing seaward through the saturated gravel *must be* given by dividing the quantity passing in a unit of time by the area of its channel. Consider a strip 1 foot in width—that is—between vertical north-south planes 1 foot apart.

The average yearly rainfall at Brooklyn is about 43.3 inches. Of this probably not far from 24 inches will sink into the ground and slowly per-

colate seaward. The entire quantity passing the line of the conduit and the driven wells from the 8 miles inland will be $8 \times 5{,}280 \times 2 = 84{,}500$ cubic feet in an average year.

The area of section through which all this must pass is 1 foot wide by 50 feet deep. Wherefore, if this were an open channel devoid of gravel the mean velocity of the water would be

$$\frac{84{,}500}{50} = 1{,}690 \text{ feet per year.}$$

In absence of mechanical analysis of this gravel for effective size and uniformity, since it is obviously of a very open, porous character, but not uniform in size, assume that in a cubic foot of the gravel 30 per cent. is voids and 70 per cent. solidity—or, in other words, that a cubic foot of the saturated gravel would contain 30 per cent. of water by bulk. The 30 per cent. of water in one particular cubic foot of space to-day will have moved forward to occupy another cubic foot of space to-morrow, and although the sectional area of the interstices measured on a plane at right angles to the direction of flow would be much less than 30 per cent. of 50 square feet, the mean velocity of translation may be considered as if flowing in a channel of 0.3 square feet net area per foot of gross area. This gives for the section 1 foot wide by 50 feet deep, a net equivalent channel area of 15 square feet, and therefore the mean velocity of transfer or flow of water past the site of the wells *must be* not far from

$$\frac{84{,}500 \text{ cu. ft. per year}}{15 \text{ sq. ft.}} = 5{,}600 \text{ feet per year.}$$

This proves that near the conduit line the water must be moving seaward at the rate of about one mile per year, or about 15 feet per 24-hour day; but this is the place of maximum velocity, for obviously as we go north the volume of water passing a given vertical east and west section rapidly becomes less as the northern border of the watershed is approached, while the depth of the saturated gravel probably lessens in smaller degree.

It is therefore probable that a given barrelful of water poured into the ground at B, 3 miles from the wells, would be at least 4 years in reaching the wells, and that any pathogenic bacteria would have been nitrified and rendered harmless—killed by starvation and suffocation—before reaching the vicinity of the wells.

There are some "ifs" in the data from which we have computed this rate of flow (for example, the uniformity of the gravel), and it would be well

to make a practical test by putting down a few wash-drill holes 200 feet apart on the line of flow, and at various depths, dose the upstream well with a few gallons of strong brine—and then take samples of water daily from the lower wells for analysis, and thus determine the speed and course of the percolation.

Third—Depth of Bottom of Well a Safeguard Against Pollution:

With a well tapping the saturated gravel at 50 feet below the surface the water may be expected to have become sterile by the time it has reached the point of withdrawal even though polluted on the surface.

Out of the mean annual rainfall of 43.3 inches assume, as before, that 24 inches, or 2 feet in depth, soaks down into the porous gravel. If the gravel contained 33 per cent. of voids then this 2 feet of rain soaking into the ground in each year would, as it slowly moved downward, if drawn off from the bottom, occupy a vertical space of 6 feet, somewhat as indicated in Figure 105.

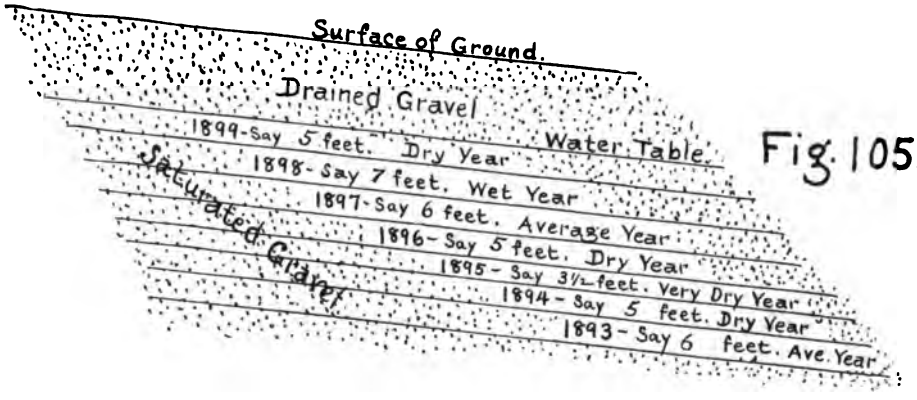

Surface of Ground.
Drained Gravel
Water Table.
1899-Say 5 feet. Dry Year
1898-Say 7 feet. Wet Year
1897-Say 6 feet. Average Year
1896-Say 5 feet. Dry Year
1895-Say 3½ feet. Very Dry Year
1894-Say 5 feet. Dry Year
1893-Say 6 feet. Ave. Year
Saturated Gravel

Fig. 105.

If, therefore, we were certain that the gravel were equally pervious at all places—and it does appear to be nearly so—we would be fairly certain that rain water falling near the line of the wells and polluted by matters on or near the surface, would either be carried by and out to sea, if the draft by the well were a small portion of the whole, or if the wells were taking nearly the entire average yield, 5 or 6 years would elapse before it sank to the level of the well-point apertures through which it would be withdrawn; and in the meantime it would have become thoroughly treated by very slow sand filtration and any bacteria taken from the surface destroyed or starved.

Considering in combination the principles stated in the two paragraphs above, and sketching the path of the rainfall on its way to the wells in Figure 106, it appears pretty nearly certain that any water reaching the wells either from near or distant points must have become thoroughly filtered on the way, and that *the health of the community will be much safer when using ground water properly collected* than when, as now, taking the main supply from surface drainage and from the surface outcrop of ground water and from open ponds in which algæ or bacteria may thrive and into which a heavy shower may wash noxious matter from adjacent roads or fields.

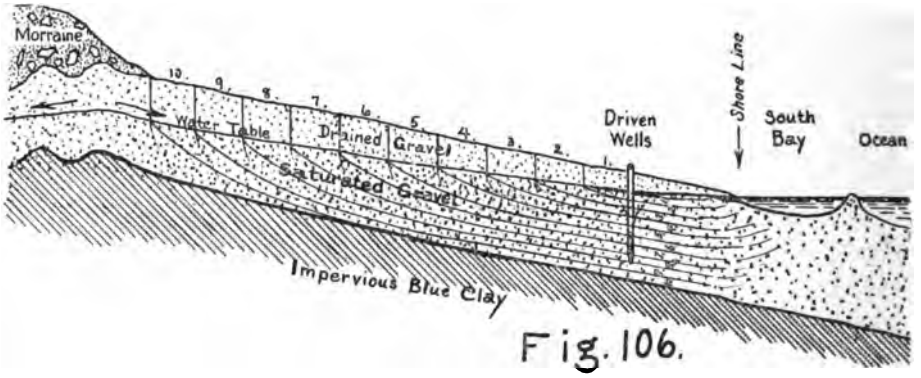


Fig. 106.

Possibility of Developing the Entire Yield of the Watershed from Wells:

By a judicious, well regulated pumping from wells driven to a level just above the blue clay and, say, 100 feet apart, more or less, it is possible to extract almost the entire subterranean flow as effectually as though one had a perfectly tight line of sheet piling or a masonry core wall extending east and west along the south edge of Long Island reaching from the surface down into the blue clay.

To make this clear consider that in Figure 107 A is 500 or 1,000 feet nearer the sea than B and that the ground water level is reduced by pumping until there is a counter slope from B to A, or, indeed, until the water merely stands level at A and B. Whenever the ground water level is reduced to this extent, then *all* flow from the north to the sea beyond the line of BC or AF *must cease*, for water with a free surface under ground, as above ground, only flows down hill.

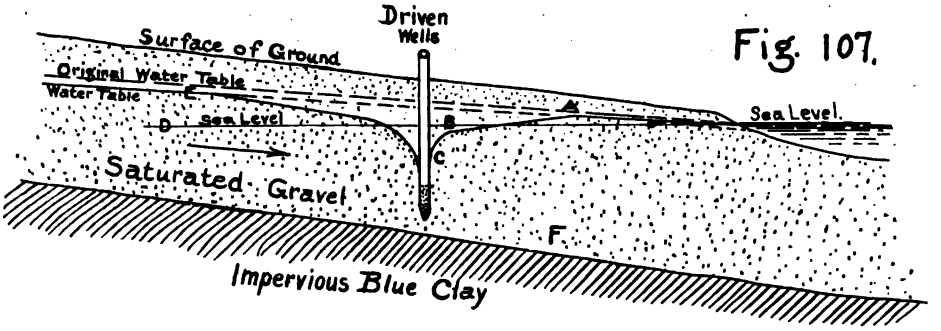


Fig. 107.

The pressure is transmitted freely through the spaces between the coarse sand grains and there can be no flow from the vertical transverse plane through BC to the similar vertical transverse plane AF, unless the head or level or hydrostatic pressure at B is greater than that at A and F.

As a practical matter it would be necessary to maintain the water level a very little higher at B than at A to avoid the possibility of brackish water flowing back under the influence of a counter slope, but this could be easily regulated by test pipes set, say, 200 feet and 728 feet away from the line of the pump wells, or one-tenth of a mile apart, and maintaining a head at the 200-foot line, say, 0.12 feet higher than at the 728-foot line, which is 1-10 mile from the first test well.

Then, since the normal slope of the ground water from the north toward the sea is here probably about 12 feet per mile, and since the velocity of flow of water in small pipes or by percolation is nearly in direct proportion to the slope, *it would be almost certain that 90 per cent. of the entire subterranean flow was being abstracted* and 10 per cent. left flowing toward the sea, or by a similar adjustment of the slope in the water table from B toward A to the general slope of the water tables north of the wells any desired proportion of the total rain water percolating toward the sea could be taken out.

Diversion of Ground Water Without Injury to Crops:

While I am personally somewhat skeptical about the real injury to crops by any such drying out of the ground as can be produced by pumping driven wells, particularly after having personally examined the excellent crops grown last season on ground near Medford, Long Island, and at other localities where the level of ground water as shown by depth of wells is said to be about 60 feet below the surface, it is a fact that under the present system of driven wells the ground water is lowered near the long lines of wells

sometimes to the extent of 20 feet, and this lowering has been carried on, I am told, in extreme cases, until a small lowering was noticed half a mile away.

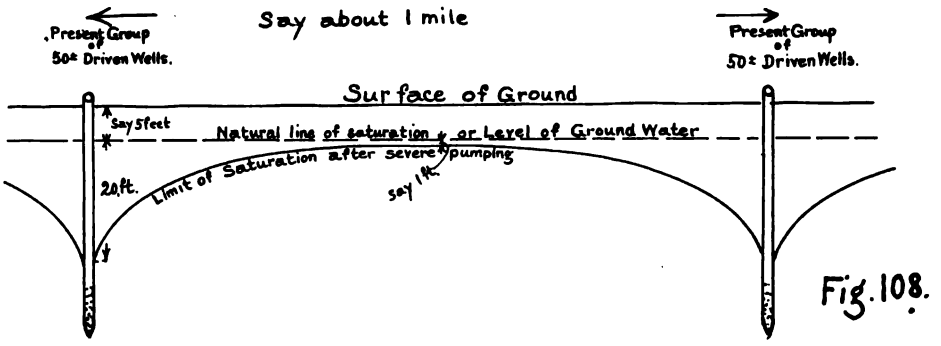

Say about 1 mile
Present Group of 50± Driven Wells.
Present Group of 50± Driven Wells.
Surface of Ground
Say 5 feet
Natural line of saturation, or Level of Ground Water
Limit of Saturation after severe pumping
say 1%
20 ft.

Fig. 108.

Vertical Section on East-West Plane.

The present condition may be represented roughly by Figure 108.

If, now, we had a continuous line of wells along the conduit, say, 100 or perhaps 200 feet apart, a much larger quantity of water could be obtained without lowering the ground water level at any point to nearly so great an extent as now, and not to an extent sufficient to injure crops. The condition in the latter case as compared with the former is represented by the two series of curves of Figure 109.

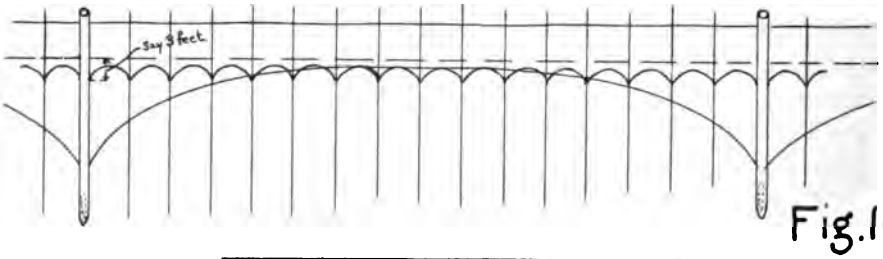


Fig. 109.

A Possible Method of Diverting Ground Water:

The foregoing consideration leads to the suggestion that it may be possible to devise a better form of conduit and a better method of diverting the ground water than that heretofore followed, one which shall better guard the public health, utilize a larger per cent. of the rainfall, and avoid heavy claims for damages to the crops.

. I therefore venture to present the following sketch, Figure 110, merely as a first study, subject to modification, of what is substantially a deep cut and cover aqueduct with a continuous line of wells feeding it by gravity flow. In the pressure of other matters I have not had time to work up the details and present this now merely to give more point to my suggestion on page 537 that the possibilities of development of the Long Island ground waters have not been exhausted and that the objections so strongly urged against driven well supplies can probably all be successfully overcome by study and experiment.

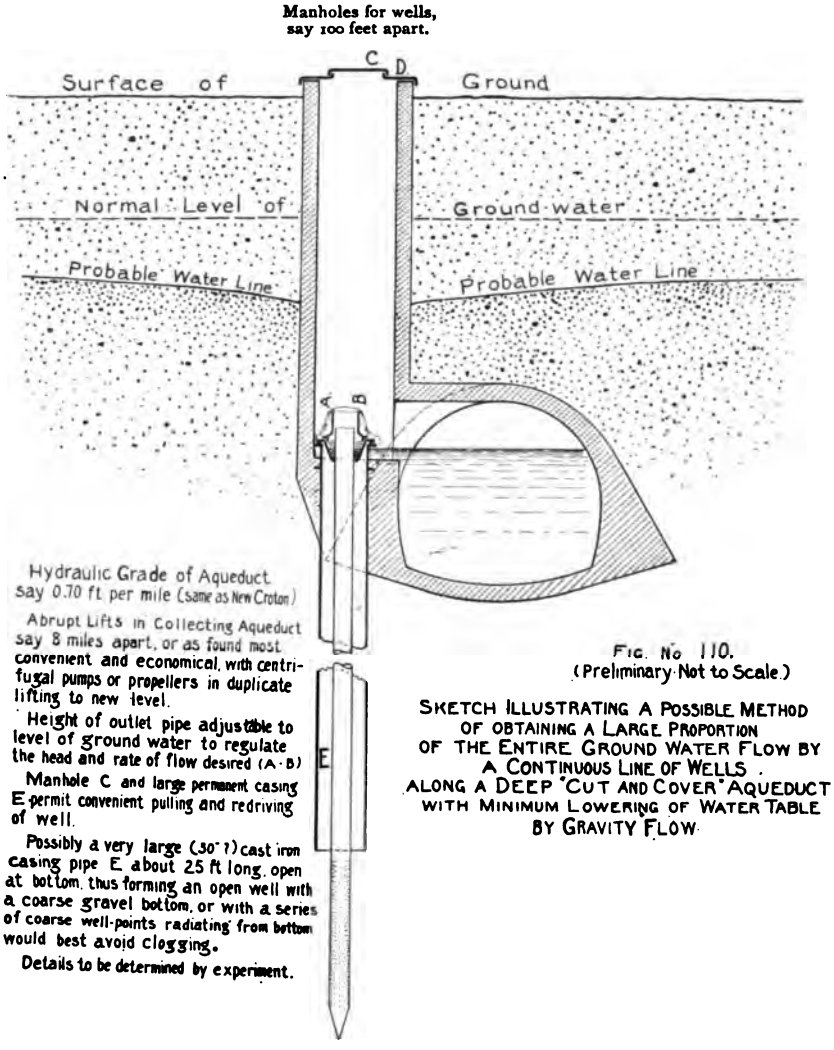

Manholes for wells, say 100 feet apart.
C
D
Surface of Ground
Normal Level of Ground-water
Probable Water Line
Probable Water Line
A
B
Hydraulic Grade of Aqueduct say 0.70 ft. per mile (same as New Croton)
Abrupt Lifts in Collecting Aqueduct say 8 miles apart, or as found most convenient and economical, with centrifugal pumps or propellers in duplicate lifting to new level.
Height of outlet pipe adjustable to level of ground water to regulate the head and rate of flow desired (A-B)
Manhole C and large permanent casing E permit convenient pulling and redriving of well.
Possibly a very large (30"?) cast iron casing pipe E about 25 ft long, open at bottom, thus forming an open well with a coarse gravel bottom, or with a series of coarse well-points radiating from bottom would best avoid clogging.
Details to be determined by experiment.
E

FIG. No 110.
(Preliminary-Not to Scale.)

SKETCH ILLUSTRATING A POSSIBLE METHOD OF OBTAINING A LARGE PROPORTION OF THE ENTIRE GROUND WATER FLOW BY A CONTINUOUS LINE OF WELLS ALONG A DEEP "CUT AND COVER" AQUEDUCT WITH MINIMUM LOWERING OF WATER TABLE BY GRAVITY FLOW.

To thoroughly investigate the possibilities of the Long Island watershed east of Massapequa, and incidentally study more fully the watersheds already in use, would require a full season's work under an appropriation of say $25,000 for wash-drill borings, chemical and bacterial analysis and engineering study to determine the watershed limits and the configuration of the surface of the blue clay floor underlying the yellow gravel; to locate the eroded valleys in it, or the possible holes in this floor through which the rainfall may leach to strata below; chemical analysis of the water from these wash-drill borings or test wells to determine the presence of iron or sulphur; and in general, a thorough survey. Obviously, I have had no such data for a guide as such a complete investigation would give, but have found sufficient data to prove that such a complete study will be well worth its cost if the Ten Mile and Housatonic sources cannot be secured, and that *some additional studies should be made* before the construction is begun on the principles and plans laid down in the Report of 1896.

This is, without doubt, one of the most remarkable, most extensive and most uniform gravel deposits in the world, and it would be difficult to imagine more ideal conditions for the gathering of a large and unfailing ground water supply. All requirements for the expense of storage reservoir is rendered unnecessary, for the interstices of the ground form a more capacious storage than is ever constructed by damming and flooding a valley and this subterranean reservoir is shielded from evaporation and from pollution, and will in all probability yield a larger safe yield per square mile of watershed than the highest line on Figure 49.

Twenty-five thousand or fifty thousand dollars expended in learning the best way to develop a supply that, by the Brooklyn estimates of 1896, would cost complete nearly $24,000,000 is surely not an extravagance, and *these same investigations would, without doubt, add to the value of the watersheds already in use.*

I may venture to add that there is good warrant to be found for recommending that a long continuous strip of land—a "right of way" 1,000 feet wide along the conduit line—should be purchased and held by the Municipality for the preservation of the purity of the ground waters entering the conduit from the driven wells. The surface of this could be slowly developed as a natural parkway containing a broad, gentle curving macadam road, like the parkways around Boston or its Metropolitan reservations, and thus far more than compensate the county for its unused waters diverted. If the aqueduct and line of wells were located, say 750 feet from the northern edge, there would be a barrier against outside pollution which (see page 540) it would take the ground water about fifty days to filter across.

BOROUGH OF RICHMOND—STATEN ISLAND.

This island covers an area of 49 square miles exclusive of the salt marsh on its borders and, according to the approximate estimates of the New York Department of Health, contains a population of 68,254. Of this total population a very considerable portion has collected in small villages which have not yet received the benefits of a public water supply. For example, the inhabitants of the thrifty villages of Pleasant Plains and Prince's Bay still obtain their water from wells like country towns, and, for example, the large and valuable factory of the S. S. White Dental Instrument Mfg. Co. at Prince's Bay is without public water for fire protection or general purposes. Indeed, with the exception of the small village of Tottenville and a small private plant at New Dorp, the south half of the island is without a public water supply.

Wells in the backyard become more and more a menace to health as the villages are prosperous and grow, and a full supply of soft, pure water suitable for domestic and manufacturing purposes is one of the encouragements toward growth, increased real estate values and prosperity which the present strong municipality appears in duty bound to furnish.

The large villages on the north end of the island are supplied by two private water companies, whose present contracts are said to expire in 1902 (see Rept. N. Y. Dept. Water Supply, 1898, p. 31), the Crystal Water Company supplying the northeast corner of the island, and the Citizens' Water Company supplying the northwest corner.

The report of the New York Department of Water Supply, 1898, page 31, states that these two companies supply in all about 6.5 million gallons per day. If this estimate is accurate it gives a surprisingly large per capita use, but this I did not attempt to investigate.

The Village of Tottenville, at the extreme south end of the island, is supplied by a small plant owned by the municipality, which, according to the Report of the New York Department of Water Supply, 1898, page 31, has a reliable daily yield of a little more than 150,000 gallons—one-sixth of a million gallons—per day.

A new 10-inch well, said to be for a reserve, was in progress in April, 1900.

When visited by one of my assistants in April, 1900, this plant was found delivering about 80,000 gallons per day (plunger displacement) and it was said that in summer this was increased to 125,000 gallons per day.

This Tottenville water was locally reported a little hard, but not unsatisfactory.

The said report further states, page 94, that all this water supply for the Borough of Richmond is "very hard" and implies that it is not of the best quality for manufacturing purposes or for use in boilers. I have not found time to examine particularly into questions of wells, pumps, or pipe distribution systems, but have confined my brief examination mainly to a consideration of the questions of extensions and of obtaining an additional supply free from pollution, the need of which is very obvious. In company with Prof. Crosby I have made a brief examination of the topography and geology of the island, questioning into the possibility of supply from local sources. For his very full answer to the questions relative to driven wells and to true artesian wells in various localities on the island reference may be had to his full report in the latter portion of Appendix 16.

The indications are that the capacity of the local sources already discovered or developed are taxed nearly to the safe limit by the population already supplied, and would be overtaxed if the pipes were extended to the villages not now supplied, and such brief examination as I have found it possible to make in the brief time allowed for this report all point very strongly to the necessity of bringing from the mainland the water needed for extension and future growth.

Proposed Supply from Mainland via Brooklyn.

My present views are that the cheapest and best means for securing an adequate future supply is to bring it across the Narrows from Brooklyn in submerged ball-joint pipe, and to establish a large reservoir between the hills on the northern part of Staten Island of such capacity that there should always be an ample reserve on hand against accident or interruption of flow through these submerged pipes. So far as yet studied it appears entirely practicable to carry water from the proposed terminal high-pressure reservoir at Park Hill, near the northern boundary of the Borough of The Bronx across to Brooklyn; thence in an almost straight line to Bay Ridge; thence across the Narrows to Stapleton or Rosebank, and thence a short distance to a distributing and storage reservoir at about elevation 200 above sea.

The arrangement of pipes for conveying this water which at present appears most feasible is to extend directly south from Long Island City one of the two steel pipes proposed elsewhere in this report as leading from the Park Hill to Ridgewood, carrying it in as nearly a straight line as possible southwesterly directly and full size to Bay Ridge, branching off on the way, by 36-inch tees to Ridgewood and Prospect Park Reservoirs, with weighted relief valves on the outlets feeding these reservoirs, thus retaining the full Park Hill pressure—less friction—in the conduit.

This large pipe would thus serve for the main artery for supply and fire protection in the most important commercial and manufacturing districts, and for the interests along the waterfront for all Brooklyn, affording high pressure water in ample volume for the most severe demand. and at the same time give an ample surplus of capacity for the supply of Staten Island and the future needs of the Gravesend district.

This main artery to be tapped only by main pipes at intervals of say ½ mile, and to be cut into blocks of about a mile in length by 36-inch gates set on reducers on the venturi principle. So that a section could be temporarily cut out and by-passed by the rest of the distribution system.

From Bay Ridge, close to Fort Hamilton, the pipe proposed would start across in water of moderate depth in which divers could readily work, and in the narrow deep central channel the depth indicated by the U. S. C. S. charts is about 60 feet and not below that at which divers can do the little work required. It is proposed to use flexible joint pipe similar to the "Falcon" pipe of about double the ordinary thickness, much of the way across the depth is less than 40 feet.

Ultimately two lines of pipe, each of say 24-inch diameter in the submarine part, should be laid, the two lines being preferably at least 300 feet apart.

There is from surface indications, and so far as one can judge without numerous test pits and borings, an excellent site for the reservoir of any desired capacity, between the hills at or near the site of Silver Lake. The surface soil appears to be remarkably impervious, as is indeed shown by the existence of this pond.

The cost of transportation of this water across to Staten Island from Brooklyn and from Park Hill depends on the proportion of the large line through Brooklyn charged to Brooklyn, but putting as much as appears fair upon Staten Island and dividing this by the quantity that would naturally be provided for in the capacity of these pipes in view of present and prospective population, the cost per million gallons would be moderate, or not exceeding $ per million gallons.

Necessity for Bringing New Supply from Brooklyn or Mainland.

The reasons for this are so plain that the explanation may be brief. Brief consideration will, I believe, cause anyone to admit that the four methods described below are the only practicable methods or sources for securing a water supply of the magnitude that the public needs.

First—Storage of surface water, in reservoirs which collect and retain the storm water plus the "spring water," or ground water which falling in rain is stored in the interstices of the soil between the sand grains and slowly percolates toward a lower level until it breaks out in springs.

Second—Pumping from driven wells or pipes driven to 30, 50 or 80 feet below the surface, to tap the underground water coming from the rainfall that is stored in the interstices of loose sand or gravel formations or is slowly percolating seaward through the upper strata.

Third—Pumping from deep wells or true artesian wells, thereby obtaining water from deep strata, which probably soaked in at some very distant point and is slowly percolating seaward or toward a lower level, tightly covered by impervious strata, above the porous stratum, thus passing, it may be, beneath salt water and closely held down by the strata above.

Fourth—Transportation across the Arthur Kill or Kill van Kull, or across from Brooklyn.

Surface Water Supplies on Staten Island Insufficient.

A day's driving over the island, or the study of a good topographical map, like that in Bien's Atlas of the Metropolitan District, scale 2 inches per mile, or the topographical map of the U. S. Geological Survey, scale about 1 mile per inch, will prove beyond question that the watersheds are too small in area, too much cut up and the streams too short and too small to give any

hope of an adequate quantity. There are 10 such, little brooks and valleys, on the island, exclusive of those now utilized, but the entire 10 only cover a total drainage area of about 8 square miles. The dam sites on these streams give no outcrop of ledge, but are on apparently deep deposits of an impervious and rather soft bowlder clay, which is excellent as regards its impervious quality, but is a material that would erode so rapidly, if once overtopped or perforated, as to make very high dams impracticable.

There are perhaps 3 or 4 locations where little brooks leading from watersheds of 1 to 2 square miles could be impounded, and by pumping the water to give it additional pressure could supply little communities of 2,000 to 5,000 inhabitants; but although these particular watersheds are not now thickly settled, they would rapidly become more and more subject to pollution, and it is, after brief consideration, plain beyond question that the sum total of all of the surface supplies that the topography will admit of impounding, would quickly become inadequate to the needs of the growing communities, and would soon, if not from the very start, require filtration for safety against pollution, while most of the sites would require pumping in a small and comparatively expensive way. The possibility of any local surface supply may therefore be quickly dismissed.

Additional Driven Well Supplies for Staten Island.

The present supplies are all from driven wells. While the best locations have doubtless already been occupied and appear to have been pumped to the full safe limit of a dry year's yield, there are a very few other sites for wells similar to those at Tottenville, 40 to 100 feet deep, that might be developed and temporarily piece out the supply for a small village.

The conditions for obtaining any considerable supply from driven wells on Staten Island are very unfavorable. This water here can have no other origin than local rainfall. Almost the entire island is covered with a very impervious blanket of clayey drift that hinders the absorption of the rainfall. The subterranean drainage areas are not large, and the deposits of porous strata in which ground water may be stored from a wet year to a dry year are not large. The rainfall is sometimes low.

The Brooklyn rainfall, nearly equi-distant from the coast, per page 48, Table 5, Report on Additional Water Supply for Brooklyn, 1896, was—

32.00	inches	in 1864.
32.76	"	in 1863.
38.02	"	in 1862.
32.71	"	in 1856.
33.41	"	in 1848.
32.69	"	in 1845.
35.64	"	in 1895.

The total evaporation, as shown by difference between rainfall and run-off, averaged 23.14 inches for the whole 32 years of the Croton measurements. The total evaporation may be smaller in a dry year because of there being less wet surface exposed to sun and wind, but plainly the margin left to supply ground water in the relatively small deposits of porous gravel and sand is not sufficient.

From a study of the possible gathering ground and from the imperviousness of the sandy clay that covers almost the entire island, it appears almost certain that an addition of, say, 50 per cent. to present aggregate yield, by seeking new sources for shallow wells on the island would be impossible, and the water from present wells, will become open to suspicion from pollution as the territory over the drainage becomes more thickly settled.

Eighty per cent. of the area of Staten Island is covered by the terminal moraine of the continental glacier. The surface formation is very impervious, a typical bowlder clay, soft and sandy—in fact, more sand than clay—but which, as is shown by the way water stands in every little hollow among the hills for a long time after a rain, or apparently until evaporated, does not, like the yellow gravel of Long Island, let the rain fall down early into the substrata. It is thus difficult or impossible to tell how much water reaches the substrata from the surface, and impossible without extensive test-wells to know where are the boundaries of the subterranean drainage area.

To illustrate the apparent limitations of the present supply, we may take the clove valley as an example. It was stated to me by the man in charge of the pumps at the Crystal Water Company's Clove Pumping Station, that in sinking their driven wells, this sandy clay was found to be but 15 or 20 feet thick and that deep below this was a stratum of gravel free of clay, from which their water is drawn, but their pumps are in the middle of a broad hollow between the hills and there very likely is a rim of ledge around the deeper part of the basin, nearly surrounding this gravel, and few, if any, equally fortunate sites can be found on the island.

This site has a watershed amounting to. .....1.29 square miles
as shown by the surface, and now is being drawn on at the
rate of about.....600,000 to 700,000 gallons per 24 hours
and in summer is drawn on to the extent of.....1,000,000 gallons per 24 hours
or sometimes.....1,200,000 gallons per 24 hours
according to the statement of the man in charge.

This is about all one could expect to continue to draw permanently from a watershed of only 1.29 square miles with an impervious steep surface and comparatively little foliage, and with the small annual margin between rainfall and evaporation noted on the previous page.

Impossibility of Any Large Artesian Supply on Staten Island.

Some persons are always hopeful about deep subterranean waters and "underground rivers."

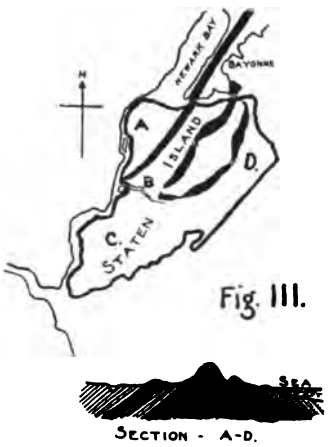


Fig. III.

The fact of many good wells across in New Jersey gives to one not familiar with the geological conditions governing these matters some warrant for hope in well-sinking on Staten Island. Therefore, I asked Prof. Crosby to review this question in the light of these New Jersey wells with especial care. His report will be found in Appendix No. 16.

All subterranean water on Staten Island must be of local origin, for the territory is hopelessly cut off from any extension of pervious strata from the mainland.

Referring to the outline sketch of Staten Island in the margin, or better, to the geological map of Staten Island, Figure 112, it will be seen that there are two impassable barriers in the line of any such subterranean flow of water from New Jersey toward the

northern two-thirds of Staten Island. The first barrier being a continuation of the great dike of trap forming the Palisades of the Hudson, which prolonged, is the foundation on which rest Bergen Point and Bayonne, and which, although it disappears below sea level at Bull's Head at B, continues farther south. The second barrier is the great eruption of serpentine, a primitive rock more anciently in place than any possible deep gravel stratum through which water might pass; this second mass of impervious rock presents an unbroken barrier all the way from Tompkinsville to the great marsh at Fresh Kills, and borings at Perth Amboy indicate that it continues farther south at comparatively shallow depth.

This leaves only the small marginal strips at A, B and C in which fresh artesian water from the mainland could by any chance be found.

At A the deep strata, mainly of sandstone, slope backward to the north and west, or in the wrong direction to convey it. At B, the serpentine barrier probably continues south at no great depth below sea level, for this is indicated by borings at Perth Amboy, and thus also cuts off the territory C from any deep previous connection with the corresponding cretaceous strata of the New Jersey mainland in which water has been found.

Reference is to be had to Prof. Crosby's report for a fuller discussion of these matters; and while it cannot be said that a supply of 50,000 or possibly 100,000 gallons per day, or sufficient for a large factory, might not be obtained at some of these points (for a supply of this size might be of local origin although obtained from a deep bed of sand or gravel), *any large volume of fresh water suitable and sufficient for a public supply from any deep wells anywhere on Staten Island is in the highest degree improbable.*

The practical experience with the deeper wells driven on this island has been, as would be naturally expected, either a well entirely unsuccessful or yielding but a small flow.

As this borough becomes more populous the only course left, is the bringing of water across either from New Jersey or from Brooklyn. If from New Jersey, it must apparently of necessity be furnished by a private company, and one can hardly expect a cost less than that charged several of the neighboring cities for their supply in bulk. While by taking this as a part of the much larger development required for New York, and with the advantage of the low rate of interest that the city can secure, *it would appear certain that the final cost from the municipal source via Brooklyn would be only half or two-thirds as much as from a private company in New Jersey.*

Appendix No. 16.

OUTLINE OF THE GEOLOGY OF LONG ISLAND IN ITS RELATIONS TO THE PUBLIC WATER SUPPLIES.

BY W. O. CROSBY.*

*The writer's opportunities for original observations on the geology of Long Island have been comparatively limited; and this summary, especially as regards the statements of fact, is therefore to a large extent a compilation from the writings of other students in this field, including especially Bryson, Dana, Hollick, Lewis, Merrill, Ries and Upham.

INTRODUCTION.

The leading facts in the geology of Long Island having a bearing upon the distribution, movement and availability of the ground waters are:

First—The formations slope seaward 20 to 40 feet per mile across the entire breadth of the island.

Second—They consist, from the surface downward, of:

(1) The yellow gravel, usually 50 to 100 feet thick;

(2) The blue clay, also 50 to 100 feet thick;

(3) The gray gravel 10 to 100 feet thick; and

(4) A great thickness (500 feet or more) of alternating clays, sands and gravels belonging to the Cretaceous formation and resting upon the fundamental cyrstalline rocks (granite, gneiss, etc.).

Third—The chief water-bearing strata are the yellow gravel the gray gravel and certain beds of gravel and sand in the Cretaceous series.

The main purpose of this study is to determine what light our present knowledge of the geological structure of Long Island throws upon the following problems:

(1) Is it possible to obtain a copious supply of water from deep wells on Long Island, 200 feet or more in depth, passing through the blue clay into the gray gravel and the still deeper water-bearing strata of the Cretaceous? A supply of quality suitable for domestic purposes and in quantity sufficient for a substantial addition to Brooklyn's water supply, say 10, 25, 50 or more, million gallons per day, or the equivalent of the yield of a watershed of 10, 25, 50, or more, square miles?

(2) What certainty or probability is there that wells 40 to 80 feet deep, sunk in the yellow gravel but not penerating the blue clay, in the region east of Massapequa, can be made to yield water of suitable quality for domestic

supply and in quantity equal to the total average catchment of rainfall on a watershed of the area shown by the surface topography to be tributary to the proposed point of taking?

(3) Is there any geological reason to expect that the yield of ground water available per square mile will be materially different in quality or in quantity from that in the region already developed west of Massapequa?

(4) Is there any apparent advantage in one location over another for tapping the subterranean waters of the yellow gravel?

Although, in the absence of many essential facts, it is impossible to give specific and definite answers to all of these questions, certain general answers or conclusions are indicated which it is believed should be regarded in developing plans for an increase in the water supply of Brooklyn. These conclusions, as well as the facts upon which they are based, are fully set forth in the following pages, but they may be briefly summarized here.

In regard to deep wells, it is undoubtedly true that while here and there a deep well may tap a supply of good, potable water sufficient for the supply of a small village or a large factory, or a supply yielding, perhaps, 50,000 to 100,000 gallons per 24 hours, *the geological formation gives no hope of finding any large permanent deep-well supply sufficient in volume to form a substantial increase to the water resources of Brooklyn, or adding materially to the volume that can be obtained from shallow wells.* There is no reasonable possibility of any large volume of water finding its way over to Long Island from the mainland by percolating through deep, porous strata, and whatever yield of fresh water comes from these deep wells must have its origin in rain falling upon Long Island.

There seems to be no reason to doubt that the water afforded by the yellow gravel in the region east of Massepequa would compare favorably in quality with that from the same formation west of Massepequa; but it is highly improbable that the yield of water per mile of conduit extension would be so large.

A line extending approximately east and west along the south shore of the island, back from the sea just far enough to avoid the indraft of water rendered brackish by sea water, is obviously the best line on which to locate wells, because of the southward dip of the water-bearing yellow gravel and its impervious floor of blue clay.

The surface valleys cannot be relied upon to give indication of the depressions or valleys in this clay floor of the water-bearing gravel which

The Water-bearing Strata.

The water-bearing formations or horizons of Long Island are practically limited, as previously stated, to the yellow gravel above the blue clay, the gray gravel below it and certain layers of gravel and sand in the underlying Cretaceous formation; and, as will be clearly set forth in what follows, those are named here in the order of their value and availability. The yellow gravel, which receives and holds almost the entire rainfall of the island, ranks first in importance; while the gray gravel and the Cretaceous gravels, being cut off from any possible influx of water from the mainland of Connecticut or New York, can afford but a limited and uncertain supply, which is likely to be stale or mineralized, and, if heavily drawn upon, to become brackish.

Geological History of Long Island.

In order to understand the arrangement and succession of the strata and to estimate the probability of copious and permanent supplies of water of good quality for domestic use being found in any particular stratum, we must study the history of the formations and the conditions under which their component strata were laid down.

Origin of the Lowest or Cretaceous Strata.*

In late Jurassic and early Cretaceous times southern New England stood higher than now; and the Connecticut river, following the belt of red (Triassic) sandstone and shale southward to the present site of New Haven, continued seaward across a lowland which is now occupied in part by Long Island. During Jurassic and Cretaceous times the surface of this region was being slowly base-leveled or reduced to the condition of a peneplain, partly through the agency of marine erosion during a slow subsidence of the land, and the Cretaceous sediments, chiefly clays and sands, with beds of gravel in the lower part, were deposited to a thickness of 500 and possibly 1,000 feet over the site of Long Island, reaching far inland over a large part of southern New England. There were, doubtless, many oscillations of level. During movements of subsidence, the sea, actively wearing away the sinking land, formed banks of gravel and sand near the shore and deposited the clays and finest sands farther out in deep, still water; then, during movements of elevation, the gravel and coarse sand were swept seaward and spread over

* To give at least a roughly approximate idea of the time required for the formation of these and the succeeding strata, it may be stated that, according to what appear to be the best estimates of the length of geological time, the duration of the Cretaceous period was several millions of years; and at least two and possibly three millions of years have elapsed since its close.

the clays; and thus, by many repetitions of this process, the alternating series of coarse and fine sediments was gradually built up.

The land was then slowly elevated, exposing the newly formed Cretaceous sediments as a broad, smooth coastal plain across which the sluggish streams wandered seaward without regard to their deeply buried original channels. It thus happened that the Connecticut, below Middletown, abandoned its natural course as determined by the north-south belt of Triassic sandstone, and wandered off to the southeast, reaching the ocean across what is now the east end of Long Island, west of Montauk Point. The Housatonic experienced a similar diversion to the southeast; but the Hudson adhered to its original north-south course to its present mouth at the Narrows, whence its channel may be traced by soundings southeastward across the sea-bottom for a hundred miles. As the Cretaceous strata were gradually worn away, the streams cut down through them into the crystalline rocks; and they necessarily held to these new courses after the Cretaceous beds had been completely removed from what is now the main land. This explains the fact that while north of Middletown the Connecticut meanders through a wide north-south sandstone valley, south of Middletown it abruptly leaves this valley and flows through a narrow gorge in the crystalline peneplain on which it was superposed in a purely accidental or fortuitous manner by the wearing down, during the period of elevation, of the Cretaceous plain across which it had pursued the most direct seaward course.

Origin of the Tertiary Strata, Including the Gray Gravel, the Blue Clay and the Yellow Gravel.

The erosion of the Cretaceous formation was followed by a second slow but less extensive submergence of the land, during which were deposited the Tertiary strata. As the Tertiary sea slowly encroached upon the land, the Cretaceous strata remaining above sea level were worked over by the waves and the coarse material sorted out and deposited always near the advancing shore to form a basement bed of coarse, dark gray gravel and sand, while the clayey element was deposited farther out in deep, still water to make the blue clay formation overlying the dark gray gravel and sand. This dark gravel and sand formation, which, although an important water horizon, is more or less interstratified with clay, is commonly 10 to 20 feet thick, but sometimes much thicker (50 to 100 feet) where it fills hollows in the eroded Cretaceous beds. As stated above, the blue clay was being laid down at the same time with the gray gravel, the clay off-shore and the gravel in-shore, during a period of subsidence, so that as the sea encroached more and more

on the land and wore down and worked over the earlier deposits, the gravel first laid down was covered later by the clay as the shore line moved farther in. The overlying blue clay is normally at least 100 feet thick; but in some places is much thinner, where it has been eroded before the deposition over it of the great mass of iron-stained quartz gravel the formation of which is explained in the next paragraph.

Finally the shore line, in its northward progress, reached the hard crystalline rocks (granite, gneiss and schist) of the recently denuded Cretaceous peneplain of southern Connecticut, and the subsidence of the land beneath the waves was so slow and long-continued that the sea, co-operating with subaerial agencies (rain, frost, streams, etc.), had time to develop, for a breadth of 5 to 10 miles, a new (Tertiary) peneplain several hundred feet lower than the Cretaceous peneplain. During this slow process of peneplanation (wearing down of the land to an approximately plane surface) the sea also had time to reduce all the constituents of the crystalline rocks, except the quartz, to fine sand and clay, which were swept seaward by the waves and currents and deposited in deep water remote from the land; while the resistant quartz, once in the form of narrow white veins in the crystalline rocks, but with its matrix long since eroded, the veins themselves broken and crumpled by frost and worn by flowing streams and further worn and rounded by the waves of the ancient sea, was left in the form of well-rounded pebbles or gravel, near the shore, covering the newly-formed Tertiary peneplain. The gravel is thus essentially a residuary product of long continued marine erosion; although it probably consists, in part, also of the worked-over Cretaceous gravels, which had been similarly derived from the crystalline rocks during the development of the Cretaceous peneplain and sediments.

This slow subsidence of the land was probably followed by an oscillation (elevation and subsidence) during which the blue clay suffered some erosion and the great mass of quartz gravel was washed seaward and spread over the eroded surface of the Tertiary blue clay as we now find it, in a bed having a normal thickness of about 75 to 100 feet, but attaining a greater thickness up to 300 feet, where it fills hollows or valleys eroded in the blue clay. This immense body of residuary quartz gravel, which is now the principal formation of Long Island, covering all the plain country north and south of the terminal moraine, and which, by virtue of its superficial position and open (pervious) structure, is also the chief reservoir of the ground waters, has become more or less stained with iron oxide and is known by geologists as the yellow gravel. From the western end of Long Island the yellow gravel extends southwestward across Staten Island and the lowlands of New

Jersey; and it is to be regarded as one of the most important as well as one of the latest formations of the Atlantic seaboard. It belongs chiefly, at least, to the Lafayette division of the Neocene period; while the underlying blue clay and gray gravel may be referred, provisionally, to the Chesapeake division of the Neocene.

Origin of the Terminal Moraine, the So-Called "Back-Bone" of Long Island.

Finally came the important and long-continued elevation of the land which ushered in the great Ice Age, the total upward movement amounting, according to the evidence of the deep submarine channel of the Hudson river, to at least 3,000 feet. During this elevation of the land, the lateral tributaries of the Connecticut, Housatonic, etc., wore back the northern or landward edge of the bed of yellow gravel and the underlying clays and sands, developing the abrupt slope or escarpment which now forms the northern edge of Long Island—a typical cuesta overlooking the inner lowland which is now occupied by Long Island Sound.

As the land gained in height, the great ice-sheet slowly crossed the lowland, pushed against the cuesta or northward-facing escarpment, distorting the strata, and finally surmounting the cuesta, built upon its crest the terminal moraine, which is largely composed of yellow gravel, clay and sand, scraped up by the ice from the northern face of the ridge.

Description of the Terminal Moraine.

The moraine is, in places, several miles wide, and ranges in height from nothing to about 200 feet, presenting, where most typically developed, a strongly undulating surface—a disorderly group of winding ridges, hummocks and hollows, with many closed basins or "kettles."

The moraine, known commonly as the "backbone" of Long Island, is most normally developed toward the west end of the island, in Brooklyn, where the till or bowlder clay attains its maximum thickness of 150 to 200 feet, and thence eastward to Roslyn, or as far as the Sound is narrow. As the Sound widens eastward we find that the proportion of material which was brought across from the mainland by the ice-sheet steadily diminishes; and the moraine is composed in increasing proportion of sand and gravel scraped up by the ice from the northern or scarp face of the cuesta, with occasional bowlders, sometimes of great size (10 to 30 feet in diameter), resting upon it.

The deep fiord-like indentations or bays of the northern coast all have a south-southeast trend, coinciding with the direction of glacial movement; and there can be no doubt that they were formed chiefly, if not wholly, by the scouring action and thrust of thick lobes of the ice-sheet. It is of special interest in this connection to note that the highest and broadest parts of the moraine lie in the trend of these fiords, or just where the theory that the fiords were plowed out by the ice would lead us to expect a special accumulation of material. Thus, south-southeast of the indentations of the coast named below we find the corresponding hills:

Hempstead Harbor.....Harbor Hill, 384 feet high.
Cold Spring Harbor.....Janes Hill, 383 feet high, and other hills above 300 feet, including the ridge extending south to Bethpage.
Huntington Bay.....Dix and Comac Hills, 250 feet.
Smithtown Harbor..... Mt. Pleasant, 200 feet.
Stony Brook Harbor .....Bald Hills, 200 feet.
Port Jefferson Harbor.....Reuland's Hill, 340 feet.
Wading River.....Terry's Hill, 175 feet.

That the clay ridge or ridges extending south-southeast from Cold Spring and Huntington Harbors to Bethpage and West Deer Park are due to glacial thrust appears reasonably certain, the overlying gravel having been largely scraped off by the movement of the ice.

The southern limit of the ice action is very indefinite, being marked chiefly by scattering bowlders. While the ice-sheet was building its moraine, the water resulting from its melting partly escaped to the southward, washing more or less sand and gravel down across the northern edge of the broad plain of yellow gravel extending from the moraine to the southern coast. This over-wash material or apron plain is probably relatively unimportant, and we may regard the main body of the gravel forming this great plain as pre-glacial, and as extending continuously through the morainal tract to the northern coast. In fact, the moraine itself is chiefly due to the mashing up and scraping up of the gravel as previously stated. Figure 17 is a generalized section of the moraine, showing its relations to the underlying strata.

Post-Glacial Deposits and Changes.

Since the recession of the ice-sheet the land has subsided until the sea has covered the inner lowland, forming Long Island Sound, and it is supposed that this part of the coast, after one or more oscillations, is still sinking.* By virtue of the monoclinal structure, the strata all dipping to the

* The coast in this vicinity is supposed to be sinking at the rate of about 2 feet in a century. This matter was very carefully investigated by Prof. George H. Cook, late State Geologist of New Jersey, and his conclusions appear to be almost universally accepted or at least undisputed.

south 10 to 25 feet per mile, the northern coast of the island is bold and sharply defined; while the southern coast is shifting and indefinite and bordered by a continuous line of barrier beach.

The recent studies of glacialists afford fairly accordant indications as to the length of post-glacial time, the most probable estimate being that something like 10,000 years have elapsed since the final disappearance of the great ice-sheet from the latitude of Long Island. The only noteworthy changes in the geological formations of Long Island during this post-glacial period are the construction by the waves of the barrier beach along the southern coast, the erosion of the headlands of the northern coast, and the partial silting up of the bays, harbors and inlets of both coasts. The extent of subaerial erosion during the same period has been almost inappreciable, owing largely, as explained in the next paragraph, to the high porosity of the yellow gravel, which prevents the formation of surface streams.

The preservation of the integrity of the plain of yellow gravel south of the moraine and forming from one-half to two-thirds of the entire area of the island, through glacial and post-glacial times, is undoubtedly due to the well assorted and consequently highly porous character of the material, which absorbs the rainwater and prevents the formation of surface streams above the general water level of the plain. Where the gravel is so situated that it can be undermined by the washing away of the underlying clay, the gravel must, of course, suffer erosion with the clay. But over its upper surface the gravel is almost impregnable, so that its geological age is out of proportion to the slight amount of erosion which it exhibits.

The yellow gravel must have been, when first deposited, white or non-ferruginous, for the mechanical action of the waves would soon remove the pelicle of iron oxide which now invests each pebble; and the present iron-stained condition of the gravel is an index of its porosity, being clearly due to the percolation and evaporation of meteoric water, probably at a time when the land stood higher than now and the water-level may have sunk in dry seasons to the bottom of the gravel, giving the atmosphere a chance to penetrate and peroxidize the dissolved iron, one source of which is, perhaps, to be found in the pyritiferous blue clay, especially where it has been scraped up to form a part of the terminal moraine.

Summary of the Geology of Long Island and Correlation of the Formations.

Regarding Long Island as it now exists, we find that it is a sloping plain from 10 to 20 miles wide and from 100 to 200 feet high along its deeply indented northern edge. On this high edge is built the terminal moraine, adding from zero to 200 feet to the height. The crystalline rocks are exposed only on the extreme northwest corner of the island, in Long Island City and the immediate vicinity of Hell Gate. The surface of the crystalline rocks slopes to the southeast, and probably at a somewhat uniform rate, gaining a depth of about 550 feet below sea-level in the first five miles, as shown by the artesian well at Woodhaven.

Upon this crystalline floor rests the great thickness of Cretaceous sands and clays. The Cretaceous outcrop is very largely covered by the terminal moraine; but its southeastern border is a line extending, as shown on the map, from Bay Ridge, on the Narrows, to Eaton's Neck. The outcrop of the bed of dark gray sand and gravel separating the Cretaceous and Tertiary clays and forming the lower water horizon is quite clearly indicated by the east-west indications of the shores of the northern bays, from Manhasset to Eaton's Neck. The intervening headlands or necks are thus more or less completely divided, the Cretaceous clays outcropping on the northern member and the Tertiary clays on the southern member of each neck. Beyond Eaton's Neck, however, the Cretaceous clays are wanting, and in the vicinity of Port Jefferson and on Crane's Neck we have only the Tertiary clays.

The southern border of the Tertiary outcrop, as the map shows, passes near the head of each bay or harbor. In consequence, however, of the disturbance of the strata due to the pressure and thrust of the ice-sheet, the Tertiary clay also outcrops south of this line at East Williston, Bethpage and West Deer Park.

Near Brooklyn the normal dip of the strata is southeast and farther east it is south-southeast. The dip evidently exceeds the slope of the broad coastal plain (10 to 20 feet per mile), averaging, perhaps, 25 feet per mile. From a comparison of the Woodhaven and Barren Island artesian wells, Woolman concludes that the normal dip in that part of the field is south-southeast, about 45 feet per mile.

The Cretaceous formation is a complex of coarse and fine sediments—a constant alternation of sand and clay, with some gravel, through a great thickness. The blue clay is in part distinctly carbonaceous (black) and characterized by carbonized wood or lignite, and both disseminated and nodular sulphide of iron (marcasite). Superficially, also, the clay is often oxidized to buff and brown tints.

The fact that on Long Island the outcrops of the Cretaceous and Tertiary strata are almost completely covered by the terminal moraine, and the further facts that these formations are still unconsolidated and that in the morainal belt they have been very extensively disturbed and displaced by the pressure, thrust and abrasive action of the ice-sheet have baffled all attempts to unravel the details of the stratigraphy and obtain the data for a satisfactory correlation with the formations of New Jersey by surface observations. It is obvious that for the solution of this problem we must depend chiefly upon the records of artesian wells and other borings; and these are as yet insufficient for the purpose. Greensands, which are such a prominent feature of the Upper Cretaceous of New Jersey, are conspicuous by their absence on Long Island. This fact, with others of like import, and the absence of distinctive fossils, indicate that the Upper Cretaceous, at least above the Clay Marls, is, in the main, unrepresented on Long Island. This may be due to non-deposition, but quite as likely to the erosion which preceded and attended the advent of the Tertiary strata; for the borings indicate very plainly a strong erosion unconformity between the Cretaceous and Tertiary.

That the blue clay, with its intercalated and underlying sands and gravels represents the Chesapeake division of the Neocene Tertiary is reasonably certain; but it does not appear that the diatomaceous phase of the clay is well or distinctly developed on Long Island. The yellow gravel which rests in entire unconformity upon all the older formations, may, in the main, at least, be referred with confidence, as in New Jersey, to the Lafayette division of the Neocene; while the modified drift south of the moraine, consisting to a large extent of the yellow gravel and not always readily distinguishable from it, but with incorporated subangular pebbles of granite and gneiss, and other glacial detritus, is clearly the equivalent of the Columbia division of the Pleistocene.

Relations of the Ground Water to the Geological Structure.

Water-bearing Formations of the Atlantic Seaboard.*

AGE.	FORMATION.	THICKNESS.
Pleistocene	Glacial drift, including the Terminal Moraine	
	Columbia, or glacial gravel south of the moraine	
Neocene	Lafayette—Yellow gravel	
	Chesapeake—Marls, clays and sands, and the great Diatom bed	
Eocene	Shark River—Part of Upper Marl bed	12 feet.
Upper Cretaceous	Manasquan—Part of Upper Marl bed	65 "
	Rancocas—Middle Marl bed	50 "
	Monmouth—Red sand	50 "
	Monmouth—Lower Marl bed	100 "
	Matawan—Clay Marls	275 "
Lower Cretaceous	Raritan—Plastic clay	347 "
Jura Trias	Red shales and sandstones, with trap sheets	

PRINCIPAL WATER HORIZONS OF NEW JERSEY.*

First Horizon—The Raritan Group. This may be defined as a group of two or three horizons in heavy gravel and cobble strata near the base of the plastic clays, which clays are variously colored white, yellow and red, or they are mottled in all these colors. The color of the water yielding gravels may be defined as a yellowish white in contrast with those of the next higher horizon, which are of a bluish white.

Second—The Sewell Horizon. At the top of the plastic clays and base of the clay marls. The water-bearing gravels are often very coarse. Their color is a bluish white in contrast with those of the first group.

Third—The Woodbury-Wenonah Horizon. This horizon is near the base o the clay marls. In the region about Woodbury the depths of the various wells seem somewhat too irregular to indicate a well-defined water-bearing sand, and this horizon is there probably distributed through a considerable thickness of laminated sands, while in the neighborhood of Wenonah the horizon, according to our present information, is not so thick, and yet appears to be more decidedly a single sand bed.

Fourth—The Cropwell Horizon. This horizon is midway in the clay marl or Matawan formation. It is not, so far as yet known, more than a few feet thick, but yields mostly a satisfactory quality of water, often doing so at points where the water from the next higher horizon, as is occasionally the case, proves somewhat too irony.

Fifth—The Marlton Horizon. At the top of the clay marls and base of the true greensand marl series, which consists of the upper, middle and lower marl beds.

Sixth—The Lindenwold Horizon. A few wells have been developed in this horizon, which is within the Bryozoan limesands next above the middle marl bed.

Seventh—The 950-foot Atlantic City Horizon. This horizon may be regarded as one not yet thoroughly defined. The only wells so far known that can draw from it are one each at Winslow and Berkeley, and one to the 950-foot horizon at Atlantic City. This horizon is *probably* at the base of the Neocene and top of the Eocene beds.

Eighth—The 800-foot Atlantic City Horizon. About 150 feet higher than the preceding one, and about 125 feet below the base of the great 300 to 400-foot Diatom bed in the Neocene.

* Compiled from the Reports of the Geological Survey of New Jersey.

Ninth—The 700-foot Atlantic City Horizon. About 100 feet or more higher than the fifth, and a little below the base of the great Diatom bed in the Neocene.

Minor horizons yielding much less water than those above listed occur above the ninth or 700-foot Atlantic City horizon, in or in close connection with the great Diatom clay bed of the Chesapeake division of the Neocene strata.

Tenth—The Yellow Gravel Horizon. Of very exceptional importance on Long Island.

Passing along the coast northward from Barnegat, N. J., some of the upper of these horizons would not be met with at all points, but southward from Barnegat if a well were drilled anywhere sufficiently deep it would probably find all of them, unless perhaps a few of the thinner ones, which have as yet only been developed in the central and western side of the coastal plain, might not continue as open sands so far seaward.

The following table gives, where known, the approximate interval in feet from the top of one horizon to the top of the next:

WATER HORIZONS IN STRATIGRAPHICAL ORDER, COMMENCING AT THE TOP.

Yellow Gravel.

700-foot Atlantic City horizon—Interval about 100 feet.....	Neocene.
800-foot Atlantic City horizon—Interval about 150 feet.....	
950-foot Atlantic City horizon—Interval not yet known.....	Eocene.
Lindenwold horizon—Interval about 50 feet.....	Cretaceous.
Marlton horizon—Interval about 115 feet.....	
Cropwell horizon—Interval 125 to 150 feet.....	
Woodbury-Wenonah horizon—Interval about 70 feet.....	
Sewell horizon—Interval not yet determined.....	
Raritan horizons—A group.....	

The principal horizons near the coast in northern New Jersey are the Woodbury-Wenonah and the Cropwell. The southward increase in the depths of these horizons is shown by the following data:

	CROPWELL HORIZON.	WOODBURY-WENONAH HORIZON.
Asbury Park.....	400 feet.	525 feet.
Ocean Grove.....	400 "	525 "
Belmar.....	500 "	650 "
Sea Girt.....	600 "	735 "
Bay Head.....	710 "	885 "
Mantoloking.....	790 "	922 "

Both of these horizons are in the laminated sands (sands alternating with clays) forming the upper part of the clay marl formation. The clay marl is 275 feet thick, and is separated from the Triassic by 347 feet of plastic (potters') clay and lignite of the Raritan formation, the basal member of the Cretaceous.

The Raritan and clay marl formations are probably the only divisions of the cretaceous represented on Staten and Long Islands, and the clay marl only imperfectly.

THE WATER HORIZONS OF LONG ISLAND.

Water in the Deep Strata of Long Island Must Be of Local Origin and, therefore, of Limited Quantity.—Long Island is almost as completely isolated geologically as it is geographically. The fact that its sedimentary formations do not outcrop on the mainland of Connecticut or New York makes it entirely impossible that they should derive water from the rainfall of those regions, either directly or, considering the massive structure of the underlying crystalline rocks and still more the continuous barrier presented to the westward by the great trap sheet forming the Palisades of the Hudson and extending south through Jersey City, Bergen Point and Staten Island to Perth Amboy in New Jersey, by deep subterranean flow.

Southwestward the Cretaceous and Tertiary formations of Long Island are undoubtedly continuous with those of Staten Island and New Jersey, and with no barrier intervening, save the sea; but the movement of the ground-water along the strike of the strata for so great a distance is in the highest degree improbable. We must, therefore, conclude that the rainwater falling on Long Island is the only possible source of the fresh water contained in its strata.

Water Horizons in the Cretaceous Formation of Long Island.—In New Jersey, where the Cretaceous strata outcrop wholly on the land, some of the sand and gravel layers are very important water horizons—in fact, a principal source of the artesian supplies. But on Long Island these coarse strata appear to be less persistent, and they outcrop mainly below sea-level. They thus lack an appreciable hydrostatic head, and if they were heavily drawn upon the water might become brackish. In only three of the Long Island wells of which we have authentic records is a satisfactory supply of water obtained from the Cretaceous beds.

One of these is Test Well No. 25 of the Brooklyn Water Supply, and the other two are on Barren Island, in Jamaica Bay. The latter are flowing wells, yielding fifty gallons each per minute (equivalent to only 72,000 gallons per 24 hours) of excellent water from a stratum 30 feet thick of yellowish white coarse sand and fine gravel, with clay above and below. This is supposed to belong to the top of the plastic clay (Raritan), though possibly to the base of the clay marls (Matawan); and in the report of the State Geologist of New Jersey for 1896 it is stated that the water horizon opened by this well can probably be relied upon to supply fresh water to many of the beaches to the eastward along the southern coast of Long Island. It is also stated that the coarse white sand and gravel found in the Woodhaven well, between 456 and 515 feet is probably the same bed as the water horizon at Barren Island (690–740 feet), and that it could probably be made to yield water at Woodhaven. The artesian well on Barnum Island (383 feet) is believed to end in the upper part of the clay marls. It affords but little water.

In spite of the few successful wells, there appears to be but little encouragement to extend borings or wells into the Cretaceous; but it does appear to be a matter of importance to discover by systematic borings the

valleys which were eroded in the Cretaceous and subsequently filled by the gray sand and gravel series underlying the blue clay.

Water Horizons in the Tertiary Formations of Long Island.—Omitting the Cretaceous, the formations of Long Island present, in a general view, two principal water horizons: the gray gravel and sand below the blue clay and the yellow gravel and sand above it.

The blue clay and the gray gravel and sand normally underlying it, but also more or less interstratified with it, appear to be merely the coarser and finer parts of the same formation; and the fact that the gray gravel and sand in not a few instances are not water-bearing is doubtless due in part to their lack of continuity where inclosed in the clay and partly to the clay which they contain. Unlike the yellow gravel, the water-bearing quality of the gray gravel cannot be depended upon, but must be determined by trial.

Through the erosion of the blue clay, material was provided to form the limited beds of clay which are encountered here and there in the yellow gravel. These are found chiefly, as we should expect, in the lower part of the gravel and in the south part of the island, as shown in Borings Nos. 22 and 25 of the Brooklyn Water Works (see Fig. 17 B). They are probably rather rarely of particular value in determining the flow of water, since they are not likely to outcrop at the surface; and we may suppose that in some cases they are so limited in area that water descending vertically through the gravel might flow over the edge of the intercepting lenses or patches of clay and so descend to lower levels.

Comparison of the Gray Gravel and the Yellow Gravel, and their possible Communication through the Erosion of the Blue Clay.—It is not improbable that erosion before the deposition of the yellow gravel has cut at some points entirely through the blue clay, so that the yellow gravel now rests directly upon the lower gravel or the Cretaceous strata. Such breaks in the blue clay would almost certainly have north-south trends, and except along such lines, if they exist, the intake or catchment area of the lower or dark gray gravel is necessarily confined to the north shore and must be conceded to be rather limited. In other words, this lower gravel is a reservoir of great capacity and with a head of, say, 200 feet, which is now full of water. But if the reservoir were drawn upon heavily the intake might prove insufficient to keep it full. An important consideration in this connection is that the intake or outcrop of this gravel crosses all of the harbors between Brooklyn and Eaton's Neck, while east of Eaton's Neck it is wholly under the sea, suggesting the possibility that if

subjected to large drafts it might in time yield brackish water; at least at some points. The intake or catchment area of the upper gravel, on the contrary, is from 10 to 15 miles wide, and since this gravel averages at least 100 feet in depth, its store of water would seem to be practically inexhaustible, for the rainfall promptly sinks into the gravel, with very little loss by either evaporation or surface drainage. Again, the normal intake of the lower gravel being on the north side of the island, where the strata are so greatly disturbed, it is by no means certain that the flow would be wholly or mainly to the south.

Limits of the Catchment Area for the Yellow Gravel.—The highly disturbed condition of the strata on the north side of the island and the infrequency of satisfactory outcrops makes the construction of an accurate or detailed section impossible. Another very uncertain factor is introduced by the strong erosion of both the Cretaceous and Tertiary clays before the deposition over them of the water-bearing gravels. This cause of irregularity probably extends quite across the island, and it is for the determination of these erosion hollows and the great depths of gravel filling them that systematic borings are especially needed.

The abruptness of the north shore and the disturbed condition of the strata, which often dip to the north, make it impossible to attach much importance to the portion of either gravel horizon north of the moraine. The yellow gravel is naturally the most disturbed, but it also has a large catchment area, which is almost wholly above sea level, while the lower gravel, although quite certainly less disturbed and dipping quite steadily to the south, has a relatively limited catchment area, and that is largely submarine. The water in the lower gravel, having no natural outlet, must be essentially stagnant, and so remain until tapped by deep wells along the southern shore. This fact, together with the abundance of wood and other carbonaceous matter and of marcasite (iron sulphide), affords an explanation of the comparatively stale and mineralized character of this water.

The Bethpage and West Deer Park ridge of Tertiary clay greatly narrows the plain of yellow gravel, and must accentuate its basal slope. This clay, which appears again near Lake Ronkonkoma, probably attains a high level near the southern margin of the moraine from Bethpage eastward. *Therefore, little reliance should be placed upon the yellow gravel north of this line* for delivering water to the southward, and *borings are urgently needed to test the thickness of the yellow gravel and the depth and gradient of the water level in it, before plans for the development of the Brookhaven Watershed are fully matured.*

The following lists of artesian wells in Long Island are not exhaustive, embracing only those of most particular geological interest. The wells designated by number are test wells of the Brooklyn Water Works. In tabulating the borings it has been found advantageous to generalize the data in order to bring out more clearly the main geological facts. The first four borings in the tabulated list (Jamaica, Jericho, Hollis and Port Eaton) represent the northern edge of the field, while all the others are near the southern margin and mostly near the conduit line where it crosses the Hempstead Plains.

Artesian Wells in Long Island.

Number and Location.	Elevation of Surface.	Lafayette.	Chesapeake.		Cretaceous.	Total Depth.	Capacity—Gallons per Minute.	Remarks.
		Yellow Gravel.	Blue Clay.	Gray Gravel.				
No. 11, Jamaica	19.2	95	95	8	...	198	Very many.	Water in gravels.
Jericho	?	147	51	12	...	210	.....	Drift, 51 ft. Yellow gravel, 96 ft.
No. 6, Hollis Sta. (*a*)	58.6	302	...	..	104	406	.....	No water below yellow gravel.
Port Eaton	4	265	...	...	...	265	Satisfactory	Salt water to 205 ft.; fresh water at 263 ft.
East New York	....	149	...	...	...	149	.....	Ended in sandy clay with lignite.
Woodhaven (*b*)	35.6	213	145	59	139	577	.....	Ended in gneiss; little water.
No. 5, Ridgewood	61.8	193	87	4	...	284	Many	Water in gravels.
No. 4, 1¼ miles S. E. from No. 5	8.6	117.4	13	9	...	139.4	"	"
Baisley's	6 7	106	50	44	...	200	Pump, 1.350	"
No. 1, S. of Baisley Pond	5.5	88	53	14	...	155	Many	"
No. 4, Jameco (*a*)	1.7	79	58	13	...	150	.....	"
No. 2, ½ mile E. from No. 1 (*c*)	7.4	84	69	104	...	257	Many	"
No. 3, ½ mile E. from No. 2	9.8	87	115	75	...	277	.....	No water below yellow gravel
No. 8, bet. Nos. 3 and 7	10	72	139	84	...	295	.....	"
No. 7, W. of Springfield Sta. (*d*)	16	65	111	...	243	419	2	"
No. 9, S. of Springfield Sta. (*e*)	10.3	110	20	16	125	271	.....	"
No. 12, W. of Rosedale Sta. (*f*)	18	130	...	...	276	406	.....	"
No. 10, N. E. of Springfield Sta	27	89	5	118	145	357	.....	"
Barren Island	High tide	260	...	...	480	740	50	Two wells, 50 gallons each per minute.
Barnum's Island (*g*)	3(?)	70	56	42	215	383	.....	No water.
Fenhurst	....	54	46	57	...	157	Many	Four wells; flow largely.
No. 22, East of Valley Stream (*h*)	17.4	82	87	...	201	370	.....	No water below yellow gravel.
No. 23, near Merrick Sta. (*i*)	16.7	78	107	...	205	390	.....	"
No. 24, near No. 23 (*j*)	16	78	122	...	...	200	.....	
No. 25, at Watts Pond Sta. (*k*)	8.2	95	89	...	147	331	.....	Sands are water-bearing.

a. The Cretaceous in this well consists, downward of gray clay, with wood, 19 ft.; fine sand, with wood, 36 ft.; fine sand and clay, with wood, 11 ft.; fine sand, with wood, 38 ft.—104 ft.

b. The blue clay is described as sandy clays; the gray gravel is really a fine gray quicksand; the Cretaceous consists of alternating beds of sand and clay with some lignite, in beds from 5 to 50 feet thick, representing both Matawan and Raritan; the well penetrated 21 ft. into the gneiss.

c. The 104 feet of dark sand and gravel is probably in part Cretaceous.

d. The gray gravel is probably included with the Cretaceous.

e. The yellow gravel is all sand; the blue clay is described as gray clay with wood and pyrite; the gray gravel as gray sand with wood; and the Cretaceous as clay and sand with wood.

f. The blue clay and gray gravel may be included in the 276 feet of alternating sand and clay, with wood.

g. Two driven wells near Barnum's Island, 97 and 194 feet deep, failed to find the bed of blue clay, which is 56 feet thick in the artesian well. This indicates local erosion of the blue clay during the deposition of the yellow gravel. The Cretaceous of this well is believed to be the Matawan or Clay marls. It contains much fossil wood in the form of lignite. The 480 feet of Cretaceous strata in the Barren Island wells represent, according to Woolman, both the Matawan and the Raritan.

h. Clay in the yellow gravel from 56 to 67 feet; the blue clay consists of sand and clay, with wood; the gray gravel is not distinctly developed; the Cretaceous consists of numerous alternations of clay, sand and gravel, with wood and pyrite. No water below the yellow gravel except a little between 329.6 and 338.6 feet.

i. Clay in the yellow gravel from 52 to 74 feet; the blue clay and Cretaceous are essentially the same as in No. 22.

j. Clay in the yellow gravel from 68 to 78 feet.

k. Clay in the yellow gravel from 53.8 to 61.8 feet; the blue clay consists of sand and gravel, 35 ft. and clay 54 ft.; and the Cretaceous consists of alternating clay, sand and gravel, with wood.

In studying these borings, the erosion of the blue clay before and during the deposition of the yellow gravel over it demands particular attention. A valley in the blue clay is thus clearly shown by borings Nos. 9 and 12; while in the Hollis well (No. 6) and the Barren Island wells erosion has cut entirely through the blue clay and the underlying gravel, so that the yellow gravel rests directly upon the Cretaceous beds.

In this connection the gray gravel and sand normally underlying the blue clay are to be counted as a part of it, and in some cases, perhaps, where the clay is wanting, as its full equivalent.

ADDITIONAL WELLS ON LONG ISLAND.

Bulletin 138, *U. S. Geol. Survey.*

Clear Creek—300 feet—Cretaceous.
Cold Spring Harbor—125 feet—Probably gray gravel. Flows 18 gals. per minute.
Deer Park—136 feet—Water rises to 12 feet below the surface.
Farmingdale—111 feet—Water rises to 104 feet. Probably from the gray gravel. Yields 35 gals. per minute.
Farmingdale—1 mile from last—142 feet, 35 gals.
Far Rockaway—210 feet—40 gals. Probably from the gray gravel.
Freeport—285 feet—No water—Cretaceous.
Glen Head—398 feet—65 gals.—Cretaceous? Rises to 80 feet below surface.
Hempstead, near ocean—300 feet—Poor water—Cretaceous (?) Good water at 123 feet.
Hempstead, reservoir—360 feet—Flows 10 gals. Cretaceous (?).
Mill Landing—200 and 235 feet—Flow—Gray gravel (?).
Lawrence—205 feet—35 gals.—Gray gravel (?). Water from 40 feet in yellow gravel.
Little Neck—175 feet—Fair supply.
Long Island City Gas Co.—100 feet—75 gals. Yellow gravel (?).
Lynbrook—180 feet—No water below 40 feet.
Oyster Bay—125 to 190 feet—Flow 18 to 20 gals. Gray gravel (?).

Port Jefferson—120 feet—10 gals.—Water rises to 114 feet.
Roslyn—210 feet—100 gals.—Water also at 74 feet.
Near Roslyn—360 feet—40 gals.—Cretaceous (?).
Whetley Hill—310 feet—None, owing to quicksand—Cretaceous (?).
Willets Point—400 feet—None—Cretaceous (?).

The data are obviously insufficient to define satisfactorily all the erosion troughs or valleys in either the Tertiary or Cretaceous clays; and the need of a systematic exploration of the coastal plain by wash-drill borings is apparent.

The deep well at Port Eaton on the north shore is of special interest. The water level is at the surface, which is only four feet above sea-level. The well is 265 feet deep and *wholly* in the yellow gravel and sand. Salt water was obtained at intervals to 205 feet, and fresh water was first found at 263 feet.

This is a most striking instance of the disturbance and displacement of the strata by the ice-sheet. The ice is estimated to have been anywhere from a thousand feet to a mile in thickness, and no doubt its dead (vertical) weight as well as its thrust was an important factor in squeezing out the blue clay and depressing the yellow gravel and sand. The dip of the beds must be to the north here, and the great depth at which the first fresh water was found would seem to show that the movement of the ground water is northward for a considerable distance back from the shore. This well, therefore, strengthens the general conclusion that the northern watershed cannot be depended upon to furnish water to the yellow gravel south of the moraine.

Continuity of the Water Bearing Yellow Gravel Beneath the Moraine.

That the yellow gravel is continuous under or underlies the moraine there can be, as stated, no reasonable doubt; and it is probably the chief source of the water afforded by wells located on the moraine, as in Brooklyn and Long Island City, where the records show that the wells pass through from 50 to 150 feet or more of bowlder clay to a coarse, clean gravel yielding good and copious supplies of water.

The details of a few of these wells are quoted from the New Jersey Reports for 1897 and 1898:

Five wells in the Twenty-sixth Ward of Brooklyn gave: Loam and fine sand, 10 feet; water-bearing gravel, 25 feet; clay, 1 foot; sand and gravel, 35 to 65 feet; and then clay (probably the blue clay). The collective yield of these wells is 2,000,000 gallons in 24 hours. The yellow gravel is, evidently, the surface formation here. The well at Malcolm's Brewery, in Brooklyn, 62 feet deep, presents similar conditions.

Four wells in Long Island City, 100 to 135 feet deep, passed through bowlder clay to a coarse gravel, which yields from 75 to 125 gallons of good water per minute. This is probably the yellow gravel beneath the moraine.

In the following wells the yellow gravel is wanting, having, apparently, been scraped away by the ice-sheet:

Artesian Well in Calvary Cemetery, Brooklyn.

Drift (Moraine)	139	feet.
Greenish Earth (Glauconitic?), Cretaceous	39	"
White Clay	4	"
Gneiss	400	"
	582	feet.

Yields 70 gals. per minute.

Artesian Well of the Nassau Gas Light Company, Williamsburg.

Drift (Moraine)	75	feet.
Blue Clay, with pebbles	27	"
Oyster-shells, underlain by water-bearing quicksand	0.5	"
	102.5	feet.

OUTLINE OF THE GEOLOGY OF STATEN ISLAND IN RELATION TO THE PUBLIC WATER SUPPLY.*

BY W. O. CROSBY.

* The geology is compiled chiefly from papers by N. L. Britton and Arthur Hollick in the Annals and Transactions of the New York Academy of Sciences; and the artesian well data from these and the reports by Woolman in the Annual Reports of the Geological Survey of New Jersey.

INTRODUCTION.

The main purpose of this study is to determine what light the geological structure of Staten Island, so far as it is known, throws upon the problem as to the abundance and quality of the ground water and its availability for the supply of the present and prospective population of the island. The general result is decidedly negative and unfavorable; for the investigation inevitably leads to the conclusion, for the island as a whole and for the individual formations of which it is composed, that the available ground water is utterly inadequate to meet the proposed demands.

The great mass of serpentine rock forming the elevated central portion of the island is one of the tightest and driest formations known; and its structure is wholly unfavorable to the collection into definite channels of such water as does penetrate it.

The red sandstone and shale of the Triassic formation, which underlies the lowland northwest of the serpentine hills, are in part of decidedly impervious character; they outcrop on the island largely below sea level; the steep dip of the strata would require very deep wells to intercept the water of even the limited catchment area which the island affords, and the dip being wholly to the northwest makes it impossible to suppose that they could derive water at any depth from the mainland, especially as it would have to pass through the thick and continuous sheet of trap rock which, farther north, forms Bergen Hill and the Palisades.

Borings in the vicinity of Woodbridge and Perth Amboy, New Jersey, show that both the trap sheet and the ridge of crystalline rocks represented by the serpentine hills are continued to the southwest at very moderate depths below the surface, and thus the Cretaceous strata underlying the entire lowland south of the serpentine ridge, and which farther south, in New Jersey, embraces several good water horizons, are hopelessly cut off from any catchment area on the mainland, or even on the northern part of the island; while the fact that all the wells penetrating the Cretaceous strata are absolute or practical failures indicates that the water-bearing strata outcrop on the island only to a very limited extent, if at all.

The gray gravel and blue clay formation of Long Island appears to be wholly wanting on Staten Island, and the yellow gravel, which over the main part of Long Island is such an important reservoir of the ground water, has only a very limited development on Staten Island, being practically confined to the Cretaceous lowland south of the serpentine hills, where it is the chief source of the water of such wells as are not entire failures.

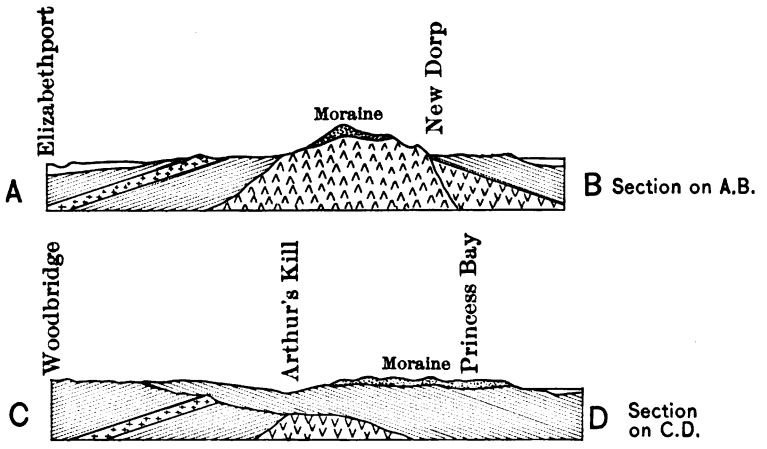
Finally, almost the entire area of Staten Island is covered by bowlder clay or drift of an exceptionally impervious character, as proved by the almost innumerable ponds, more or less effectually sealing the catchment areas of strata that might otherwise be water-bearing. Over the Cretaceous strata and yellow gravel of the southern lowland, extending from northeast to southwest for the entire length of the island, is the special development of the bowlder clay in irregular ridges and hills, which are known collectively as the terminal moraine of the great ice sheet, and in which the bowlder clay is commonly from 50 to 200 feet thick.

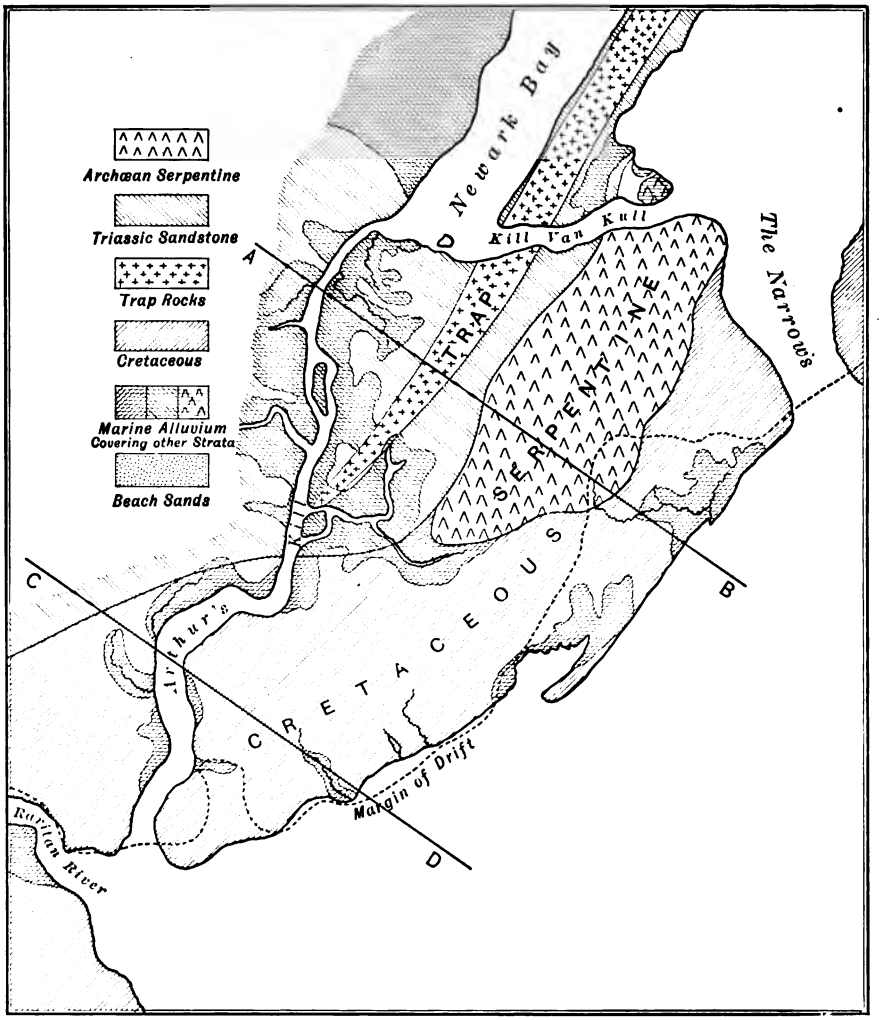
Thus, to summarize, the geological conditions affecting the storage and flow of the ground water are about as unfavorable as they could be, for the formations are only to a limited extent such as might be water-bearing under the most favorable conditions; these possible water horizons have only limited catchment areas, wholly confined to the island, and these are almost completely covered or plastered over by the impervious bowlder clay.

General Geology.

The dominant topographic feature of Staten Island is the broad area of elevated, hilly land, with many summits and crests from 200 to nearly 400 feet above the sea, which extends from New Brighton southwest six miles to Richmond. This highland, which appears to be composed wholly of serpentine rock, separates the northwestern lowland, which is crossed in a northeast-southwest direction by a low ridge marking the outcrop of the Triassic trap-sheet from the southern lowland which, extending the entire length of the island, is underlain by the Cretaceous strata and yellow gravel and specially diversified by the terminal moraine—a complex belt of drift hills and ridges from 50 to 750 feet high. Between Garretsons and New Dorp the moraine rests upon the serpentine hills, as shown in the section, and there is but one belt of hills.

The Serpentine Rock—The serpentine of the northern hills covers an area of 13.5 square miles. It is essentially a massive, impervious formation of indefinite thickness, and for the most part rather thinly covered with drift (bowlder clay). The serpentine is supposed to be a highly altered form of





GEOLOGICAL MAP of STATEN ISLAND, (modified from Britton) **Fig. 112**

some basic igneous rock, forming a great stock or boss in granitic and schistose rocks, such as now occur on Manhattan Island, but the latter are not now exposed on Staten Island, being covered at all points by the newer formations. The eastern and southern boundary of the serpentine area, for practically its entire length, is marked by an almost continuous line of very steep slopes, which testify to the resistant nature of this rock.

The Triassic Formation.—All that part of Staten Island northwest of the serpentine area, embracing 14.5 square miles, is occupied by the Triassic formation, which consists chiefly of red sandstones and shales dipping at low angles (10°–30°) to the northwest. These sedimentary rocks are divided, however, by the southern extension of the great Bergen Hill trap sheet, the outcrop of which forms a low ridge from one-fourth mile to one-half mile in width, extending from Port Richmond to Linoleumville.

The Cretaceous Formation.—The entire area of the island south and east of the serpentine, amounting to 28.5 square miles, is, so far as known, underlain by the Cretaceous strata, which, as on Long Island, have a nearly horizontal dip to the southeast, or away from the great escarpment against which they were deposited. They consist of variously colored clays and sands in many alternations, and they are a direct continuation of the Raritan or Plastic Clay division of the Cretaceous series of New Jersey, lying at the base of the formation, and below the principal water-bearing gravels and sands; although it is not impossible that the Raritan is overlain near the southeastern coast, as on Long Island, by a partial development of the Matawan or clay marls.

The Tertiary Formation.—The Tertiary gray gravel and blue clay (Chesapeake) of the Long Island series is wholly wanting on Staten Island; but the Cretaceous formation is immediately overlain by the yellow gravel (Lafayette). This is undoubtedly the best water-bearing formation on the island. Its thickness, however, is very irregular, and its surface exposure or outcrop is very limited, because it is covered almost everywhere by the terminal moraine, or over-wash modified drift from the moraine. The yellow gravel undoubtedly once covered the entire area of the island; but it is now practically confined to the southeastern lowlands, although traces of it still remain on the highest serpentine hills.

The Terminal Moraine.—The extreme southern margin of the moraine is indicated by the dotted line on the Map No. 112. Borings have shown thicknesses of bowlder clay in the moraine of 50, 80 and 200 feet. Northwest of the moraine all the upland is covered by bowlder clay, which is prevailingly

of a decidedly clayey and impervious character, thus making possible the numerous ponds.

Extensive deposits of light colored sand occur along the edges of the salt meadows on the western side of the island, representing sand dune formation before the development of the marshes.

RELATIONS OF THE GROUND WATER TO THE GEOLOGICAL STRUCTURE.

The Water Horizons of Staten Island.

It appears from the foregoing that all the formations of Staten Island, except the yellow gravel, are naturally of little value as sources of ground water; but even if the case were different, the covering of bowlder clay would cause the precipitation to run off mainly in surface streams and thus greatly reduce the intake of the underlying formations. The yellow gravel itself has, on account of the drift, only a limited catchment area, although this is greater than the map shows, as the modified drift over the yellow gravel would not prevent the surface water from descending into the latter, but would rather add to its storage capacity. Unfortunately, too, the catchment areas of the yellow gravel are mainly near the southeastern margin of the island, which would still farther limit the probability of water in the part of the yellow gravel under the moraine.

Staten Island may be compared geologically only with the northwestern end of Long Island, in the vicinity of Brooklyn, where the moraine has its strongest development and most clayey character, with only a very narrow and imperfect representation south of the moraine of the broad plain of water-bearing strata which is, after the moraine, the most striking topographic and geological feature of Long Island.

In the following paragraphs the formations are reviewed with reference to their water-bearing value:

Water in the Serpentine Rock.—The serpentine, although forming the most elevated and rugged part of the topography, is the most impervious and generally unreliable water-bearing rock on the island. We have a record, however, of one boring upon the serpentine hills which passed through 50 feet of glacial drift and 150 feet into the serpentine and is said to yield "sufficient" water. A well at Bischoff's Brewery, on the eastern margin of the serpentine, passed through 80 feet of glacial drift and 130 feet into the serpentine; but the yield is not reported. The impervious character of the serpentine would lead us, if it were covered by modified drift or gravel, to expect a good, though shallow, water horizon at the base of the gravel,

but it is covered almost everywhere by rather exceptionally stiff and impervious bowlder clay, and only a small fraction of the precipitation can sink into the ground.

Water in the Triassic Strata.—We pass now to the Triassic formation, covering the northwest portion of Staten Island, or all that part of the island directly west of the serpentine. It may be noted, first, that although the sandstone and shales of this formation are reported to supply an abundance of good water to certain artesian wells on the mainland, as at Woodbridge, Bonhampton, Rahway, Elizabeth and Newark, many wells in these strata have also proved more or less complete failures; and it does not appear that any definite and reliable water horizons have been discovered in the Triassic. In fact, none such are possible which would be available over any considerable area; for the rather high dip of the strata, commonly 10 to 20 degrees to the northwest, would in a short distance carry a water-bearing stratum beyond the reach of wells drilled for economic purposes. Since the dip is always to the northwest, and therefore *directly away from* the highlands of Staten Island, it is impossible that wells sunk in this formation on the island could benefit by any catchment area on the mainland. Nearly half the Triassic area of Staten Island is covered by salt marsh, and the remainder is divided midway by the great trap sheet, hundreds of feet in thickness. No water can be expected from the trap, and since its outcrop is a ridge, it prevents the precipitation on the higher parts of the island from flowing over the sandstone west of the trap. The best chance for artesian wells in this formation would seem, therefore, to be in the belt of sandstone and shale, half a mile to a mile wide, between the serpentine and trap, which is in a position to benefit through surface flow, by the precipitation on a part of the serpentine area, the best location for wells being, obviously, near the southeastern margin of the trap. Unfortunately, no data are at hand of wells along this line. A well at Castleton Corners, on the thin edge of the Triassic formations where it overlies the serpentine, passed through:

Earth	33	feet.
Hardpan and shale rock	53	"
Soapstone (serpentine)	64	"
	150	"

The water rises to 63 feet from the surface and yields 8 gallons per minute by pumping (or only 11,520 gallons per 24 hours). This seems to prove

that the flow from the serpentine area toward the Triassic is unimportant. A well in Bayonne, north of Staten Island, and southeast of the trap sheet, was sunk 600 feet into the Triassic red shales and obtained only a very small supply of water, a result which might have been anticipated, since the catchment area is here mainly below sea level.

Water in the Cretaceous Strata.—Although the Cretaceous strata of Staten Island are probably limited to the Raritan or Plastic Clays, this is a formation which on the mainland embraces several good water horizons—persistent beds of sand and gravel which outcrop in the latitude of Staten Island and are tapped by artesian wells at increasing depths to the southeastward. The records of artesian wells compiled by Woolman and published in the reports of the Geological Survey of New Jersey, show, however, that the Raritan is, as we should expect, a more reliable water horizon to the southeast, where covered by later formations, than on or near its outcrop. There is no chance for it to receive water from the underlying Triassic sandstone, because that dips in the opposite direction—to the northwest. On Staten Island the Cretaceous (Raritan) strata lie, on the northwest, against the steep wall of serpentine; and farther west it is specially cut off from the mainland, not only by Arthur Kill, but by the Triassic trap sheet, which passes beneath the surface at Linoleumville, and is proved by borings to underlie Woodbridge, in New Jersey, at a depth of only 56 feet, and probably extends still farther to the southwest. It can not receive water from the mainland to the southwest, along the strike of the beds, because the movement of the water will naturally be in the direction of the dip of the strata. Again, the Cretaceous of Staten Island is nearly all covered by the moraine, which, as previously noted, is mainly of a decidedly clayey character; and, as on Long Island, the formation has been so extensively disturbed and displaced by glacial thrust and abrasion that it is in the highest degree probable that the outcrops or intake areas of some of the water-bearing strata have been more or less completely covered and closed by the spreading over them of the inclosing clays. Furthermore, the Cretaceous beds of Staten Island must lie almost wholly below sea level, being deeply covered by the moraine and yellow gravel; and such water as penetrates it will, in the main, lack the hydrostatic head essential to an artesian flow. It is not surprising, therefore, in view of all these considerations, to find that as a source of water the Cretaceous of Staten Island is far inferior to that of the mainland south of Raritan Bay.

A well east of the serpentine, at the eastern end of the island, passed through 200 feet of drift (moraine) and 400 feet into the Cretaceous clays

without finding a satisfactory supply of water. A boring at Kreischerville, in the western end of the island, passed through the drift and into the Cretaceous to a depth of 196 feet, without finding water; and another well in the same vicinity, with a like result as to water, gave the following record:

Sand and Soil	30	feet.
Blue Clay (Tertiary ?)	90	"
White Sand	2	"
Sand and Clay alternating (Cretaceous)	78	"
	200	"

A recent well in the vicinity of Hugenot is reported to have penetrated the Cretaceous upward of feet without finding water.

The well of the S. S. White Dental Co., at Prince's Bay, 147 feet deep, and undoubtedly ending in the Cretaceous, is a dry hole; and another well in this vicinity, 150 feet deep, has also failed to find a satisfactory supply of water in the Cretaceous beds.

The meagre evidence is thus uniformly negative, and offers no encouragement for the further prospecting of the Cretaceous of Staten Island for water.

In this connection we may advantageously cite the records* of two artesian wells at Perth Amboy, New Jersey, as follows:

Water-works engine house just north of Eagleswood, and 28 feet above tide level. Black clay, 55 feet; red and white clay and an inch of sand, 6 feet; red shale and sandstone (Triassic), 9 feet to granitic rock at 70 feet. This was rather soft and crumbling at first, but became very hard at 85 feet and so continued to 470 feet. and no water.

Terminus of the Easton and Amboy Railroad, 26 feet above tide level. Drift, etc., 25 feet; black clays and sands, 26 feet; light-colored clay, 25 feet; fine white sand, 11 feet; light-colored clays, 30 feet; red sandstone and shale (Triassic), 17 feet. No water below the surface gravel.

Both of these wells are in the base of the Raritan (Cretaceous) clays, and they add two more to the artesian failures under exactly the same geological conditions as obtain on the southwestern end of Staten Island. They are of special interest, however, as proving conclusively that the ridge of old crystalline rocks represented by the serpentine of Staten Island continues in the same direct line, at a moderate depth, under Arthur Kill into New Jersey. From the crest of this buried ridge the Raritan clays dip gently to the southeast and the Triassic sandstones and shales more steeply to the northwest. Wells in the Cretaceous area of Staten Island cannot, therefore, intercept water from the mainland below the level of this ridge.

* Report of the Geological Survey of New Jersey for 1885.

And considering the narrowness of the Cretaceous area on Staten Island, and the slight dip of the strata (25 to possibly 50 feet per mile), and the impervious character of the clays separating the water-bearing sands, which must effectually prevent vertical movement of the water, the futility of deep wells in this area is obvious. Below 300 feet it is manifestly a hopeless case.

Water in the Yellow Gravel.—It is probable that the successful wells on the Cretaceous area usually pass through the drift and derive their water chiefly from the underlying yellow gravel.

The reports of the New Jersey Geological Survey furnish records of such wells at Tottenville, 37, 51 and 54 feet in depth; between Tottenville and Kreischerville, 36 feet; and at Pleasant Plains, 28 to 56 feet; while a well on the moraine at Annadale 200 feet deep ended in 36 feet of yellow gravel.

It appears, therefore, that the yellow gravel can be expected to yield a limited supply of water—sufficient, here and there, for local demands, over the most of the Cretaceous plain.

THE CLOVE BASIN.

The basin holding Clove and Silver Lakes is an elevated valley in the serpentine hills. It is closed on the northeast by a broad and very perfect morainal dam to which Silver Lake owes its existence, and the tightness of which could be safely relied upon even if the basin were flooded to the crest of the barrier, some 20 feet above Silver Lake. The natural outlet of this basin is to the northwest through the Clove, a narrow transverse valley, almost a gorge, with a depth below the crest of the serpentine ridge on either side of over a hundred feet. On the southwest side of the Clove, especially, the serpentine bed rock is well covered with bowlder clay, but the indications are that the drift is not very deep in the bottom of the gorge. In fact, there is, apparently, nothing in the geological conditions, either of bed rock or drift, specially unfavorable to the construction of the dam which would convert this entire basin into a reservoir with a flow line at 200 feet; although a flow line above 180 feet would also require a small and easily constructed dam on the southeast where this narrow, transverse valley or clove cuts the main range of the serpentine hills to a depth equal to the elevation of the pass, or 180 feet. The Clove Valley is, everything considered, a somewhat remarkable topographic feature, the origin of which is probably to be sought in a stream flowing southeasterly across this area at a time when the serpentine hills were completely covered by the Cretaceous and Tertiary sediments, forming a smooth coastal plain, sloping gently sea-

ward. As this ancient stream cut down through these sedimentary deposits it encountered the resistant serpentine, into which, stimulated by a considerable fall due to the erosion of the sedimentary formations southeast of the serpentine hills, it cut a straight, narrow notch or gorge. Subsequently, when the land stood higher than now, the head waters of this hypothetical stream were, probably, drawn off by some lateral tributary of the Hudson, perhaps on the line of Kill van Kull, the downward progress of which was not retarded at so high a level by contact with the head rocks. The subsequent subsidence of the land has submerged or drowned all the deeper valleys of the region. That a slow movement of subsidence is now in progress there is abundant evidence, which has been carefully collated by Prof. Geo. H. Cook, late State Geologist of New Jersey, in his report for 1868, pages 343–373.

This hypothesis should lead to the expectation of one, if not two, deep notches in the rocky rim of the Clove basin, which should be explored by wash drill borings when considering the availability of a portion of this valley for a reservoir site.

NOTES ON THE GEOLOGY OF THE SITES OF THE PROPOSED DAMS IN THE VALLEYS OF THE HOUSATONIC AND TEN MILE RIVERS.

General Geology.

This is a region of crystalline rocks—granite and gneiss—with narrow belts of limestone and mica schist. The general trend of the formations is approximately north-northeast and south-southwest, and the principal streams exhibit in general a marked tendency to follow closely the easily eroded bands of limestone, this rock forming at most points the bottoms of the valleys. The valley of the Ten Mile river is excavated in an exceptionally broad and persistent belt of limestone trending in a general north-south direction, and the much narrower limestone band of the Housatonic valley, with its south-southwest trend, joins this at the confluence of the two rivers; and this is just the point where the Housatonic river turns abruptly to the south-southeast to forsake the limestone and pursue its devious course across the hard crystalline rocks of southern Connecticut to Long Island Sound.

These facts point very distinctly to an important diversion of the drainage, and it is probably a safe conclusion that the united Housatonic and Ten Mile drainage once continued in the direct line of the Ten Mile Valley into what is now the basin of the Croton river. The vicinity of Webatuck is the same critical point for the Housatonic river that we have for the Connecticut at Middletown, where it leaves the broad valley of soft Triassic sandstone and shale and pursues a southeastward course to the Sound across the much harder granitic rocks.

This means that during the Cretaceous period the entire region was worn down to an approximate base level or peneplain represented roughly by the crests of the existing ridges and heights of land, and this peneplain was covered at least as far north as Middletown and Webatuck by Cretaceous strata, forming a plain which sloped gently seaward—south-southeast. Across this plain, as it was slowly elevated above the sea, the rivers meandered seaward, uninfluenced by the structure of the hard rocks beneath. In a comparatively short time, geologically speaking, this cover of soft Cretaceous sediments was worn away and the rivers were let down upon the hard surface of the peneplain, and, although finding themselves out of sympathy with the geological structure at most points, they have been obliged to hold to these courses ever since. We are thus able to account for the fact that the Housatonic river turns abruptly from the belt of limestone in which its valley is excavated for many miles to the northward and cuts obliquely

through a massive ridge of granitic and gneissic rocks, the valley narrowing to a gorge as it enters the harder formation.

According to this view, the Croton watershed is only the lower part of a drainage system which once extended northward into Columbia County and into Berkshire County, Massachusetts. That the Croton Valley has been beheaded at Pawling, the present water parting of the Croton and Ten Mile drainage systems, is very obvious. We find here a broad, level north-south valley which is out of all proportion to the insignificant streams which now flow sluggishly through it and pose as its architects; and the divide, which is also the site of Pawling Village, is almost imperceptible.

The relative narrowness of the valley of the Housatonic river above its junction with the much smaller Ten Mile river, and the fact that in the Town of Cornwall it crosses an important area of granite, suggest that it may have once pursued the more direct course offered by the broad band of limestone which leaves the present valley of the Housatonic in the vicinity of Falls Village in Canaan, Conn., and extends southwesterly via Lime Rock and Sharon to the Ten Mile as the floor of a broad, continuous valley in which we now find no commensurate stream.

One of the most striking and significant features of these valleys, and especially of the broad lower valley of the Ten Mile, is the fact that the immediate valley of the river is, at most points, a comparatively narrow and steep-walled gorge cut to a depth of a hundred feet or more in the broad and approximately level, or only moderately undulating, floor of limestone from a mile to two miles wide between the main valley walls of schist and granite. This broad limestone terrace, which is developed chiefly between the contours of 400 and 500 feet and is most noticeable when viewed from its level, clearly represents a partial development in this latitude of the Tertiary peneplain. In Tertiary times the rivers wore down these limestone bands to an approximate base level, and the gorge or modern channel as clearly records the renewal of the erosive power of the rivers due to the strong elevation of the land at the beginning of the great ice age. These inner valleys or gorges are a normal phase of all the larger valleys of the glaciated zone; but in many districts, and almost universally near the coast, they are filled with glacial drift and thus obliterated as topographic features.

The fact that the gorges of the Housatonic and Ten Mile rivers are but slightly encumbered by drift, the solid bed-rock forming the immediate bottom of the channel at many points, is of very obvious importance in its bearing upon the construction of the proposed dams in these valleys.

Another fact of particular importance and interest in the history of the Ten Mile river, and especially in its bearing upon the geological conditions at the site of the proposed dam, is the distinctly retrograde course of the south branch of the Ten Mile (Swamp river) for five miles in an air line from near Wing (South Dover Station) to its confluence with the north branch or main river a mile and a half south of Dover Plains. At Wing, Swamp river is only a short mile from the Ten Mile at South Dover (see map), and yet its waters flow five miles north and then five miles south to reach that point. This extremely retrograde and roundabout character of the valley is a plain indication that the drainage has been diverted from its normal direct course by a drift barrier between Wing and Webatuck. To determine with certainty the precise course of this ancient, drift-filled channel through the dividing ridge would require a series of borings. A careful examination of the ground makes it probable, however, that the buried channel either follows the general course of the road between Wing and South Dover, keeping a little to the west of the road, or taking a more direct course from Wing to Webatuck, it follows quite closely the base of the steep southeastern slope of the valley, cutting the dividing ridge at the saddle or depression near its southeastern end.

The Ten Mile river is, in this vicinity, fully sixty feet below the level of Swamp river; but it is quite unnecessary to suppose that the buried channel is a deep gorge, dividing the ridge approximately to the level of the Ten Mile. A more probable view is that the Swamp Valley holds the relation of a hanging valley to the Ten Mile Valley, this small, northward-flowing tributary having, for obvious reasons, failed to keep pace with the main stream in deepening its valley during the preglacial elevation of this region, when the ancient, base-leveled valley floors were trenched. This unequal or differential erosion of the valley floors involved, of course, the development of a waterfall or series of rapids of gradually increasing height at the mouth of Swamp river.

The small volume of this short stream, and the fact that it has been from the beginning of its history an example of reversed drainage in a broad, flat-bottomed valley, warrants the conclusion that the floor of the valley has always been, as now, more or less of a swampy character, and that it has, consequently, been deepened more through the quiet solution of the limestone by swamp waters than through the active corrosion of its bed by the stream itself. We are thus able to explain the broad and shallow character of the part of this valley attributable to Swamp river.

When, during the melting of the great ice-sheet, its southern margin receded north of the divide at Pawling, the upper valley of Swamp river was occupied by a shallow glacial lake to which superglacial and subglacial streams were tributary and with its outlet southward at Pawling into the upper valley of Croton river. As the ice margin continued to recede northward the glacial lake increased in area and depth, and the glacial streams deposited in it large amounts of sand and gravel, to a considerable extent in the form of gently sloping delta plains. It is probable that the ice lingered in the deeper Housatonic Valley until it had disappeared from the main part at least of the Ten Mile Valley. Thus was anticipated the proposed reservoir in this valley; the drainage was once more tributary to the Croton basin and the Hudson, and the fact that the drift of the Ten Mile Valley below the level of the pass at Pawling is modified (washed and stratified) is readily explained. Above this level the drift is mainly unmodified, bowlder clay taking the place of sand and gravel. The modified drift which now encumbers and diversifies the bed of this temporary lake has a high degree of porosity and would thus add materially to the capacity of a reservoir flooding this area. The deposits of modified drift in the Housatonic Valley appear to be less extensive, and may, perhaps, be attributed to marginal glacial streams, the conditions having been less favorable for the damming of the valley by an ice barrier.

Site of the Proposed Dam in the Housatonic Valley.

The site of the proposed dam in the Housatonic Valley is about one mile south of the Village of Merwinsville, at the point where the river finally leaves the belt of limestone and, turning more to the southeast, enters a gorge in the hard crystalline rocks.

The rock—a hard biotite (black mica) granite, largely of gneissoid aspect—outcrops in massive ledges at or near the water's edge on either side of the river, and rises boldly to elevations of 500 to 600 feet above the river. On the southwestern slope, naturally, some glacial drift has lodged, and a talus of large angular blocks of granite and gneiss testifies to the steepness of the slope. It is very evident, however, that the hard and massive bed-rock comes to or very near the surface to at least as great a height as the top of the proposed dam. On the northeast side the drift is thin, and the outcrops of the granitoid rock are bold and massive, affording an ideal foundation for the proposed dam. We may be very confident that, on both sides of the river, the bed-rock is more solid and unbroken where it is covered by drift than where it is exposed, owing to the protection afforded by the drift, espe-

cially against frost action. The narrow bench or terrace on the northeast side of the valley, at an elevation of about 100 feet, appears to mark the upper limit of the modified drift (washed gravel); and there can be no doubt that below this level, especially, the bed-rock surface is water-worn, smooth and hard to the very bottom of the valley.

I see no reason whatever to doubt that the massive granite and gneiss which are of essentially the same character on the opposite sides of the river, are continuous under the river, which has simply cut a notch in a previously continuous ridge of these resistant rocks; but the depth of the bed-rock surface under the river must be, in the absence of borings, largely a matter of conjecture. The narrowness of the valley at this point suggests that the depth is probably not great; and the comparative abruptness with which the bed-rock rises from the water, especially on the southwest side, indicates that the gorge which trenches the bottom of every important valley in this latitude, and dating from the strong uplift of the land at the beginning of the glacial period, is here only partially filled with drift, and not wholly buried. The gorge is likely to be rather flat-bottomed (U-shaped), but deepest on the southwest or concave side, the maximum depth, to hazard a guess, being possibly twenty feet below the level of the river, but probably less than that. The shallowness of the gorge finds corroboration in the ledges of limestone crossing the river a mile and a half or so above the dam site, and at several other points before we reach Bull's Bridge, three miles above the dam site, where the river falls over massive ledges of limestone in a gorge averaging not more than twenty feet in depth on either side of which the limestone is well exposed across the broad floor of the valley between the bounding ridges of granite.

Near the top of the ridge, on each side of the river at the dam site, but especially on the northeast side, the conditions appear to be very favorable for opening a quarry, from which could be obtained in abundance large and shapely blocks of granite suitable for use in building a large part at least of the dam.

Site of the Proposed Dam in the Ten Mile Valley.

The proposed dam site in this valley is about half a mile above the Village of Webatuck, the valley is quite narrow and wholly in limestone, which forms a nearly precipitous bluff near the north bank of the river, and a more gradual slope on the south side. There is but little drift at this point, and the solid limestone ledge comes, practically, to or very near the surface all the way from the top of the hill on either side down to the narrow flood plain of

the river. The surface appearances are favorable to the view that the preglacial gorge is only partly buried, being represented by the existing surface contours, and the bluff indicates that the axis of the gorge is toward the north side of the valley, where the depth may be ten, or possibly twenty feet below the present bed of the river. The probable shallowness of the gorge is confirmed by the ledge on which the dam at Webatuck is built and the ledges to be seen in the bed of the river below the dam at South Dover.

The buried or drift-filled gorge of Swamp river previously referred to may lie to the south of this dam site; and, if so, it could be readily located and profiled by borings. It is a safe assumption, however, that it is filled with impervious bowlder clay, which must have a horizontal thickness or breadth, at the very least, of one-fourth of a mile; and it may, therefore, be regarded with as little apprehension as if it were solid ledge.

MASSACHUSETTS INSTITUTE OF TECHNOLOGY,
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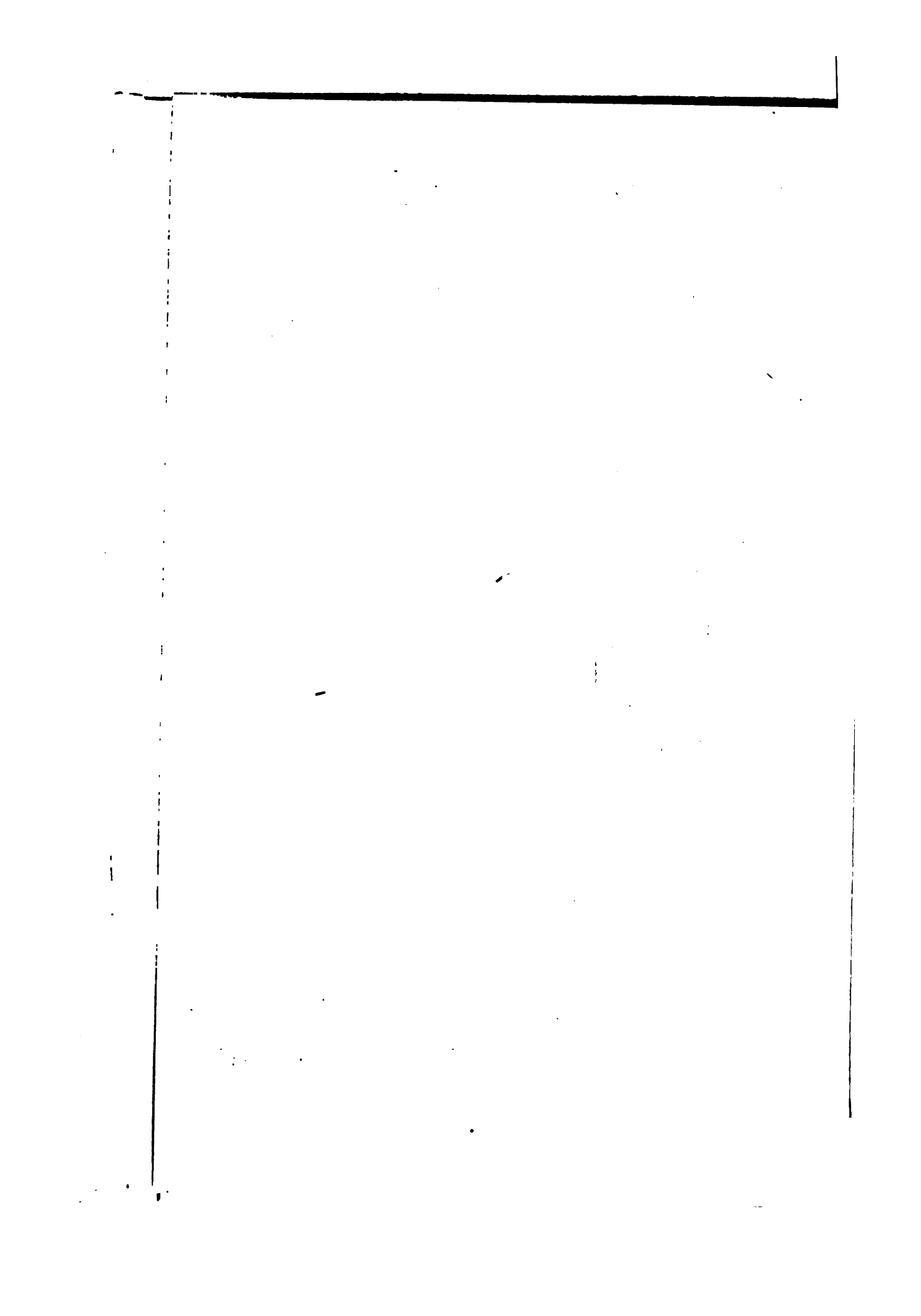

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